LAKE PONTCHARTRAIN, LA.
AND VICINITY

LAKE PONTCHARTRAIN HIGH LEVEL PLAN

DESIGN MEMORANDUM NO. 19A

GENERAL DESIGN

SUPPLEMENT NO. 1

LONDON AVENUE OUTFALL CANAL

FRONTING PROTECTION PUMPING STATION NO. 4

ORLEANS PARISH

US Army Corps of Engineers
New Orleans District

DEPARTMENT OF THE ARMY

NEW ORLEANS DISTRICT, CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DECEMBER 1994

CELMV-PE-TG (CELMN-ED-SP/23 Mar 95) (1105-2-10c) 3d End Mr. Burkhard/cc/
601-634-5930
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design - Supplement No. 1, London Avenue Outfall Canal Fronting Protection Pumping Station No. 4

CDR, Lower Mississippi Valley Division, Vicksburg, MS 39181-0080
26 JUL '95

FOR Commander, New Orleans District, ATTN: CELMN-ED-SP

The action taken in the 2d Endorsement is satisfactory. The following comment is furnished for the record.

Para 1i, 2d Endorsement. The statement that the sponsor currently has credits (LERRDs and Work-in-Kind) that total their 30 percent required contribution needs to be verified and documented. ER 1150-2-301 and ER 1165-2-131 place responsibility on Project Managers to ensure that credits for LERRDs and Work-in-Kind are in compliance with regulations. The credit amount is subject to Government audit to ascertain allocability, reasonableness, and allowability. The District Resource Management must establish cost accounting and record keeping procedures sufficient for audit. No credit will be afforded for LERRD and construction contributions that the Government has not determined to be necessary for the construction, operation, and maintenance of the project or separable element. You should also confirm that the Real Estate cost estimates are in accord with the Army Audit Agency procedures on Lake Pontchartrain, Louisiana.

FOR THE COMMANDER:

10 Encls
1. nc
wd encls 2-10

JAMES R. HANCHEY
Director of Planning and Engineering

DEPARTMENT OF THE ARMY
NEW ORLEANS DISTRICT, CORPS OF ENGINEERS
P.O. BOX 60267
NEW ORLEANS, LOUISIANA 70160-0267

REPLY TO
ATTENTION OF:

CELMN-ED-SP (1110-2-1150a) 23 Mar 95

MEMORANDUM FOR Commander, Lower Mississippi Valley Division,
ATTN: CELMV-PE-TS

SUBJECT: Lake Pontchartrain, Louisiana, and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design - Supplement No. 1, London Avenue Outfall Canal Fronting Protection Pumping Station No. 4

1. The subject Supplemental Design Memorandum is submitted for review and approval and has been prepared generally in accordance with the applicable provision of ER 1110-2-1150, dated 26 August 1993.

2. The designs presented in this Supplemental Design Memorandum were prepared by the New Orleans District of the U.S. Army Corps of Engineers. The signature and registration designation of the Chief of Engineering Division appear in this document within the scope of his employment as required by ER 1110-1-8152.

3. Approval of this report as a basis for preparation of Plans and Specifications is recommended.

FOR THE COMMANDER:

Encl
(16 cys fwd sep)

W. EUGENE TICKNER, P.E.
Chief, Engineering Division

(CELMN-ED-SP/23 Mar 95) (1105-2-10c) 1st End Mr. Burkhard/cc/ 601-634-5930
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design - Supplement No. 1, London Avenue Outfall Canal Fronting Protection Pumping Station No. 4

CDR, Lower Mississippi Valley Division, Vicksburg, MS 39181-0080
25 APR '95

FOR Commander, New Orleans District, ATTN: CELMN-ED-SP

We have reviewed the subject general design supplement and have the following comments:

a. Page 1, para 1b, Background. This paragraph should be deleted. All land acquisition and relocations associated with the parallel protection have essentially been completed by the non-Federal sponsor, and the Federal investment for design and construction is approximately 54 percent complete. Considering the sunk costs for the parallel protection plan, the parallel protection plan is now the only cost-effective solution.

b. Page 7, para 8a. For purposes of future design activities, you are reminded that the basis for structural steel design is being phased from EM 1110-2-2101 to EM 1110-2-2105, Design of Hydraulic Steel Structures.

c. Page 13, para 10e. The factor of safety for the Q-case loading under usual conditions should be changed to 1.5 to comply with the analyses shown on Plates 29 and 30. This should be corrected in your file copies of this report.

d. Page 20, para 27d and page 22, para 29d. The sections referenced here are shown on Plate 19 not Plate 24. This should be corrected in your file copies of this report.

e. Page 23, para 35, 7th line. The word "slumps" should be changed to "sumps." This should be corrected in your file copies of this report.

f. Page 25, para 37d, Table 3. Section 4-2.c of EM 1110-2-2906, Design of Pile Foundations, 15 January 1991, states that the minimum factor of safety to be used in determining the design pile capacity under an "unusual" loading condition is 1.5 if verified by pile load test and 2.25 if not verified by pile load test. This table as well as Plates 24 and 25 should be revised to reflect this.

g. Page 25, para 39, 4th line. Delete the first phi symbol after the word "arctan." This should be corrected in your file copies of this report.

h. Page 33, Table 7, Schedule for Design and Construction. The design and construction schedule does not agree with the current approved schedule shown in the Life Cycle Project Management Reporting System. If you propose to change the approved milestone dates, a request for approval along with the required justification and assessment of impacts should be submitted.

i. Page 35, Table 10, Total Federal and Non-Federal Funding by Fiscal Year. The funding schedule shown is based on incremental funding. A breakdown of costs to show the Federal and non-Federal shares is needed to facilitate development of a fully funded budget request.

j. Plate 24, Notes 5 and 6. These notes indicate that it may be acceptable for a Contractor to use either an impact or a vibratory hammer in driving piles for this project. You should ensure that the project specifications require that the test piles and the service piles be driven by the same type of hammer.

k. Plate 32.

(1) General Note No. 2. The phi symbol should be before the equal sign. This should be corrected in your file copies of this report.

(2) This plate does not indicate whether the stratum consisting of the fat clay (CH - soil type 3) was analyzed. Conversations with CELMN-ED-FD personnel indicate that these analyses were performed and yielded acceptable results. In addition, CELMN-ED-FD personnel reported that additional analyses have been conducted using a narrower neutral block and that these also yielded acceptable results.

FOR THE COMMANDER:

James R. Tuttle

Encl wd

JAMES R. TUTTLE
Acting Director of Planning
and Engineering

CF (w/encl):
CECW-EP (5 cys)

CELMN-ED-SP (CELMN-ED-SP/23 Mar 95) (1110-2-1150a) 2d End Mr. Elmer/2618
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design - Supplement No. 1, London Avenue Outfall Canal Fronting Protection Pumping Station No. 4

DA, New Orleans District, Corps of Engineers, P.O. Box 60267, New Orleans, LA 70160-0267
29 Jun 95

FOR Commander, Lower Mississippi Valley Division, P.O. Box 80, Vicksburg, MS 39181-0080, ATTN: CELMV-ED-TG

1. Our responses to the comments contained in the 1st End are as follows:

a. Para a. We do not concur. The purpose of this paragraph is to document the sequence of events leading to the parallel protection plan, events that are not documented anywhere else. Although the sunk costs for parallel protection are substantial, remaining costs for parallel protection are still greater than the gated structure cost. The remaining cost for flood proofing the bridges that cross the canal could surpass the cost for an automated gated structure at the lake end of the canal. We do propose to combine and rewrite paragraphs 1a and 1b as shown on the enclosed revised page 1 of the report (Encl 2, 16 copies).

b. Para b. Noted.

c. Para c. Concur. Sixteen corrected copies of page13 for your copies of the report are enclosed (Encl 3).

d. Para d. Concur. Sixteen corrected copies of pages 20 & 22 for your copies of the report are enclosed (Encls 4 & 5).

e. Para e. Concur. Sixteen corrected copies of page 23 for your copies of the report are enclosed (Encl 6).

f. Para f. Concur. Sixteen corrected copies of page 25 and plates 24 & 25 for your copies of the report are enclosed (Encls 7, 8, & 9).

g. Para g. Concur. Sixteen copies of page 25 for your copies of the report are enclosed (Encl 7).

h. Para h. Concur. The change in milestones will be processed through the PRB reports.

CELMN-ED-SP
SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design - Supplement No. 1, London Avenue Outfall Canal Fronting Protection Pumping Station No. 4

i. Para i. The sponsor currently has credits (LERRDS and Work-in-Kind) that total their 30% required contribution. Therefore, 100% Federal Funds will be required to complete this feature.

j. Para j. Concur.

k. Para k.

(1) Concur. Sixteen copies of plate 32 for your copies of the report are enclosed (Encl 10).

(2) Noted.

FOR THE COMMANDER:

Daniel Marcelin, P.E.

10 Encls
wd encl 1
Added 9 encls
2-10. as

W. EUGENE TICKNER, P.E.
Chief, Engineering Division

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
HIGH LEVEL PLAN
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HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
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PROJECT HISTORY

1. Background.

a. Hurricane protection for the London Avenue Outfall Canal was addressed in the report entitled "Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19A - General Design, London Avenue Outfall Canal." Two plans were presented. One plan envisioned an innovative and cost effective control structure to be constructed at the Lake Pontchartrain end of the canal. The second plan called for construction of levees and floodwalls parallel to the canal, floodproofing five roadway bridges, constructing floodgates at the Southern Railroad (SRR) bridge and constructing fronting protection at Pumping Stations 3 and 4. The New Orleans District recommended construction of the control structure and the Orleans Levee Board (OLB) accepted our recommendation.

b. Design Memorandum 19A coverage of the control structure was sufficient for proceeding with subsequent decision documents. DM 19A also provided sufficient coverage of the levee and floodwall aspects of the parallel protection for proceeding with P&S; but, the coverage of the floodproofing of the five bridges, the floodgates at the SRR bridge and the fronting protection for the two pumping stations was not sufficient for proceeding with P&S.

c. During the continuing design of the control structure, the OLB withdrew its support. Board representatives expressed a lack of confidence in the valve design envisioned for the control structure and stated a preference for the parallel protection alternative. The United States Congress resolved the issue in the FY 1992 Energy and Water Development Appropriations Act by mandating construction of the parallel protection plan.

2. Purpose. This supplement to Design Memorandum No. 19A presents the essential data, assumptions, computations and criteria used in the design of the fronting protection for Pumping Station No. 4 and is prepared in sufficient detail to provide an adequate basis for preparing the plans and specifications.

3. Pumping Station No. 4 Location and Description.

a. This pumping station is located on the east bank of London Avenue Outfall Canal at Prentiss Avenue (see plate 2). Being situated parallel with the flow of the canal, existing flood protection is provided by the earthen levee and floodwall system of the canal being linked with the foundation and building structure of the station.

b. The station consists of six pumps as follows: two 320 CFS centrifugal pumps, three 1000 CFS horizontal pumps, and one 70 CFS vertical constant duty pump. The two 320 centrifugal pumps are housed in the original pumping house for the station. The three 1000 CFS pumps and the 70 CFS vertical pump, added subsequent to the original station, are not housed.

the two 320 CFS pumps for light rain conditions, and use of the 1000 CFS pumps for severe rain conditions. The 70 CFS constant duty pump discharges through a 50 inch discharge line which runs over the canal and into the drainage system for Pumping Station No. 3. The two 320 CFS pumps discharge through a U-shaped discharge basin into the canal. The three 1000 CFS pumps discharge through concrete culverts into the canal. The station also includes a 1000 CFS siphon which draws from the drainage system for Pumping Station No. 3. This siphon is used to relieve Pumping Station No. 3 as required during severe weather conditions.

c. No structural or foundation investigations of the station were performed. No obvious major structural defects were encountered during field surveys, and no major structural problems were reported by the New Orleans Sewerage and Water Board (NOSWB).

PROJECT PLAN

4. Flood Protection Plan. The recommended flood protection plan is to construct a continuous line of flood protection, with limited impact on the existing pumping station, which will connect the existing flood protection on each side of the pumping station. The recommended plan will incorporate use of I-wall, T-wall and gated monoliths. Gated monoliths will house emergency closure gates and will be constructed in front of both the existing discharge culverts and pumping house. The recommended plan was selected after the coordination and a series of discussions, on all the alternatives considered, with the OLB and NOSWB, who support the plan. This plan is the most cost-effective and involves only modest interference with existing pumping station operations. the plan also precludes modifications to the existing pumping station foundation and floodwall for increased design loading. The NOSWB has raised some concern relative to the effect of the fronting protection on canal flows, particularly during normal and low lake stages (see Hydrology and Hydraulics section for discussions on this issue).

Gate function. The emergency closure gates will be closed only to prevent backflow through the culverts or the pumping house in the event of pump failure during a hurricane or storm surge. Normal canal stage ranges from El.0.5 to 4.0 NGVD. Standard project hurricane canal stage is at El. 11.9 NGVD. Each pump includes a ratchet mounted on the gear reducer to prevent reverse rotation of the impellers. However, the ratchet cannot be relied upon to provide protection against design canal stages. When properly engaged, the ratchet limits backflow to seepage around the impellers. For the culverts, the high point of the discharge system is at El. 8.0 NGVD, and backflow cannot begin until the canal water reaches that elevation. Gates for any failed pump will be closed once the canal stage reaches El. 8.0 NGVD. For the machinery house, the high point of the pumping system is at El. 4.57 NGVD. Gates will be closed for any failed pump once canal stage reaches El.4.57 NGVD. Note that both the 320 CFS and the 1000 CFS pumps will have positive pumping capacity, even at a canal stage of 11.9 NGVD, if operating properly.

5. Flood Protection Alternatives. Several alternatives to the recommended project plan were considered. A brief description of each alternative follows.

a. Emergency closure gates (Two variations of recommended plan). Three locations were considered for the emergency closure gates for the 1000 CFS discharge culverts. The recommended plan has the gates in front of the discharge culverts. The two variations to this plan merely shift the location of the emergency gates. A discussion of these two alternatives follows:

(1) Modification of the end of the existing discharge culverts to house the emergency closure gates. Approximately nine feet of the existing culvert and foundation would be removed and replaced. An advantage of this plan over the recommended plan is less canal discharge loss; however, disadvantages include greater costs, a greater impact on the existing facility and an increase in operational down-time at the pumping station.

(2) Replacement of the existing discharge culverts entirely. New discharge culverts would be constructed with gates installed near the canal end of the culverts. An advantage of this plan is similar to (1) above. Disadvantages include greater costs, the longest pumping station down time of the alternatives considered and the greatest impact on the existing facility both during and after construction. Construction will require extensive temporary piping and equipment relocations, and after construction, the NOSWB questions the reliability of normal pumping operations during normal and low canal stages.

b. Existing floodwall reuse. Modification and reuse of the existing floodwall, constructed integral with the pumping station slab, was determined impractical. The existing floodwall foundation could not be relied upon to resist the magnitude of loads experienced from a canal water stage of 11.9 NGVD, and would require replacement. Replacement of the foundation would require at least partial demolition and replacement of the discharge culverts for the 1000 CFS pumps. Also, the existing machinery house provides the east side flood protection for the discharge basin of the two 320 CFS centrifugal pumps. Foundation modifications for the building may be required due to stability. No further consideration was given to this plan.

c. Internal backflow prevention gates. Installing internal Backflow prevention gates at the steel-to-concrete adapter pipes for both the 1000 and 320 CFS pumps was rejected by the NOSWB based on potential problems with priming the pumps due to air leakage at joints where the gates would be installed. No further consideration was given to this plan.

DESCRIPTION OF PROPOSED STRUCTURES

6. General Description of Proposed Structures. The project plan is to construct an independent continuous line of flood protection, with limited impact on the existing pumping station, which will connect the existing flood protection on each side of the pumping station (see plates 2-5). This protection will incorporate use of I-wall, T-wall, and gated monoliths. The gated concrete monoliths will be used in front of the discharge areas of the existing pumps. The gated monolith in front of the three 1000 CFS horizontal pumps will be built as close as possible to the existing culverts. These monolith widths will be minimized resulting in the least protrusion into the canal and therefore the least flow head loss possible. Eight sluice gates will provide emergency closure capabilities in the event of pump failure. Each of the discharge culverts for the three 1000 CFS horizontal pumps will be fronted by two gates (see plates 7 & 8). The discharge basin for the two 320 CFS centrifugal pumps will be removed, and a new discharge basin, incorporating two gates at the face of the existing pumping house, will be installed (see plate 3). This arrangement will preclude the need to close gates in front of two 320 centrifugal pumps as long as pumping capacity exists. Rain water between new and old protection will be pumped into the canal.

Concrete T-wall and concrete capped I-wall will tie the new protection with the protection adjacent to the pumping station. T-wall will saddle the existing cross-canal siphon, which will remain in service at all times (see plate 6) as required by NOSWB. Gate power will be supplied by a separate 25 Hz circuit of an existing NOSWB electrical switchboard (see plate 17). All piping and miscellaneous steel modifications required as a result of the fronting protection will be designed by NOSWB and included in the fronting protection construction contract (see plate 18).

a. <u>Gated monoliths for 1000 CFS pumps</u>. One gated monolith will be used in front of the discharge areas of the 1000 CFS pumps (see plates 4, 6, 7 & 10). This monolith will house six 120" X 132" cast iron sluice gates. This structure will be reinforced concrete, top El. 13.9 NGVD, founded on steel HP14X73 piles, with PZ-22 (or equal) steel sheet pile seepage cut-off. Pier widths will be limited to the discharge culvert wall width (18") in order not to impede culvert discharge flow. The gate slots in the piers will be staggered due to the limited pier width. The gate slot offset will be minimized to reduce the monolith width. Operating floor steel deck sections, with removable handrail and grating, will be used to provide access to, as well as support for, the gate hoisting assemblies (see plate 9). Placement of reinforcing steel, embedded steel items, construction joints, and waterstop will conform to standard construction practices. Expansion joints at the monoliths ends will include 0.5 inch joint filler.

Dewatering slots for stoplogs will be provided for periodic monolith and gate inspection and maintenance. Monolith maintenance will include all required structural and cosmetic repairs, and debris removal. Gate maintenance will include functional checks and periodic replacement of the flush bottom seal.

b. <u>Gated discharge basin for 320 CFS pumps</u>. This gated monolith will be a replacement structure for the existing discharge basin (see plates 3 & 10). This monolith will incorporate two 96" X 84" cast iron sluice gates at the face of the existing machinery house. The structure will be reinforced concrete, top El. 13.9 NGVD, founded on steel HP14X73 piles, with PZ-22 (or equal) steel sheet pile seepage cut-off. Gate monolith pier widths will be sufficient width to place the gates in line. Operating floor steel deck sections, with removable handrail and grating, will be used to provide access to, as well as support for, the gate hoisting assemblies. Placement of reinforcing steel, embedded steel items, construction joints, and waterstop will conform to standard construction practices. Expansion joints between the monolith and the existing machinery house will include 0.5 inch joint filler.

Dewatering slots for stoplogs will be provided for periodic monolith and gate inspection and maintenance. Monolith maintenance will include all required structural and cosmetic repairs, and debris removal. Gate maintenance will include functional checks and periodic replacement of the flush bottom seal.

c. <u>Gates</u>. Cast iron sluice gates with electric motor-driven operators will be used based on the following:

(1) <u>Gate selection</u>. Two gate types, cast iron sluice gates and fabricated steel vertical roller gates, were evaluated. Cast iron sluice gates were selected based on initial construction cost and maintenance. See cost comparison below:

GATE TYPE	INITIAL COST	MAINTENANCE COST
SLUICE	*$61,000	NONE
ROLLER	*$58,000	**$60,000

* based on Rodney Hunt 120" X 132" gates; does not include hoisting assembly.

** includes removal, sandblasting, inspection, repainting, seal replacement, and reinstallation for a 120" X 132" gate every seven years; 50 year design life: $60,000; ignores inflation and depreciation.

(2) Operator selection. Two types of hoisting assemblies, motor-driven mechanical and hydraulic actuators, were evaluated. Motor-driven electric mechanical hoisting assemblies were selected based on NOSWB criteria. Operators will have manual back-ups.

d. T-wall monoliths. Two closure monoliths will adjoin the gated monoliths (see plates 3, 4, 5, 10 & 11). These monoliths will be inverted T-Type reinforced concrete structures, top El. 13.9 NGVD, founded on steel HP14X73 piles, with PZ-22 (or equal) steel sheet pile seepage cut-off.

e. Walkways and operating floor. The operating floor for the sluice gates will be removable steel deck sections, with removable handrail and grating (see plates 3, 4, 9). Walkways from the existing equipment deck will provide access to the operating floor of both the 1000 CFS pump gates and the 320 CFS pump gates. Walkway will also be incorporated into the stem of the T-wall monoliths to provide access to the permanent dewatering pumps.

f. I-wall monoliths. I-type floodwall consisting of steel sheet piles capped with a reinforced concrete wall will tie the existing I-wall to the gated discharge basin for the 320 CFS pumps on the south end, and to the T-wall monolith on the north end (see plates 3, 5, 10). The top elevation will be 14.4 NGVD. This elevation will be 0.5 ft higher than the pile founded monoliths to account for settlement. Steel sheet piling sizes will include the existing ARBED AZ-18, Frodingham 3NA, and new PZ-22 (or equal). The concrete portion of the floodwall will extend from El. 2.5 NGVD to El. 14.4 NGVD on the south end and from El. 2.0 NGVD to El. 14.4 NGVD on the north end. Expansion joints in the floodwall will be spaced approximately 30 feet apart, adjusted to fall at the steel sheet pile interlocks.

g. Dewatering bulkheads. Dewatering bulkheads, i.e., stoplogs, will be single, solid, aluminum panels designed to fit the gated monolith dewatering slots. The stoplog for the gate adjacent to the cross-canal siphon will consist of several panels of suitable size to be placed within the available clearances. The stoplogs will provide water retention to a canal stage of 4.0 NGVD. Permanent storage of stoplogs sufficient to dewater two gates will be provided. Equipment for handling of stoplogs will also be provided.

h. Temporary cofferdam. The construction of the fronting protection will employ two temporary cofferdams. The cofferdam design presented herein is an anchored steel sheet pile system. This type of cofferdam is simple, reliable and expensive. However, several cofferdam designs are practical, and the project construction contractor will be responsible for the actual cofferdam design used.

The first cofferdam will be installed parallel to the bank and protect the entire project work area while construction of the fronting protection is performed (see plate 16). The protrusion of the cofferdam into the canal will be minimized to limit flow head loss during PS3 (upstream) operations. The cofferdam will be divided into two phases to facilitate pumping station operations during construction. In the first phase, the south section of the project will be constructed, allowing pumping from the 70 CFS constant duty pump and the three 1000 CFS horizontal pumps. In the second phase, the north section of the project will be constructed, allowing pumping from the 70 CFS constant duty pump and the two 320 CFS centrifugal pumps.

After removal of the first cofferdam, a second cofferdam will be installed across the canal. The cofferdam will consist of a south and north wall, installed beyond the limits of the existing concrete liner, spanning from bank to bank. After dewatering, all damage to the concrete canal liner, resulting from installation of the first cofferdam, will be repaired.

NOSWB pumping operations will continue as required during construction. Both cofferdams, which will include provisions to flood the work area to allow priming of the pumps at PS4, will remain in place during pumping episodes. The heights of both cofferdams are therefore important in insuring timely construction of the fronting protection, without limiting NOSWB pumping operations in the canal sufficiently to result in local flooding. In lieu of risk-based criteria, which would employ the expected frequency of pumping events of various magnitudes from PS3 and PS4 based on pumping records, the height of the cofferdam protection will be based on the assumed practical criteria denoted below. If questionable or unacceptable flowlines result during the hydraulic analysis of these cofferdam systems, a risk-based analysis may be used to determine the appropriate cofferdam heights.

(1) For the first cofferdam, the protection must be sufficient to prevent flooding of the work are when the lake level is the annual average high (El. 4.0 NGVD) and PS3 is operating at full capacity.

(2) For the second cofferdam, the north-end protection must be sufficient to prevent flooding of the work area when the lake level is the annual average high (El. 4.0 NGVD). The protection may include a removable upper barrier, which would be installed when required and removed during pumping episodes. The south-end protection must be as high as possible without reducing the PS3 pumping efficiency to the point where the pumps cannot empty the suction basin fast enough to prevent local flooding from the suction basin. Submerged pipes connecting the south and north cofferdam walls, carrying up to 1,100 CFS from PS3 across the work area, may be used if significant benefits to PS3 pumping operations during construction are identified.

Several alternatives to the second cofferdam will be investigated during the preparation of plans and specifications. Included in these alternatives will be a portable cellular cofferdam used to perform dry repairs to the canal liner in sections, and submerged canal liner repair methods.

The top elevation of the anchored steel sheet pile design presented herein is El. 5.75 NGVD. This elevation corresponds to the flowline at PS4 with a lake stage of El. 4.0 NGVD and both canal pumping stations operating at full capacity. This elevation is excessive for the required protection and used only to determine conservative loading conditions on the cofferdam.

i. Canal liner. A reinforced concrete liner was previously installed in the discharge areas of the pumping station to prevent canal bottom erosion. The portions of this liner which must be removed during construction will be replaced upon completion of the fronting protection.

STRUCTURAL DESIGN

7. Scope. The analysis and design concepts for the structural components are presented in the following text. A general layout of the structure is presented on plate 2. Design details such as steel connection weld size and length, development and cut-off lengths for concrete reinforcement, etc. will be addressed in the final plans preparation.

8. References. The structural components shall be designed according to the applicable portions of Corps of Engineers (COE) Engineering Manuals, Engineering Technical Letters, and other reference material.

a. COE publications:

(1) EM 1110-1-2101, Working Stresses for Structural Design (Nov 63).

(2) EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures (Apr 92).

(3) EM 1110-2-2502, Retaining and Floodwalls (Sep 89).

(4) EM 1110-2-2906, Design of Pile Foundations (Jan 91).

(5) EM 1110-2-3104, Structural and Architectural Design of Pumping Stations (Jun 89).

(6) EM 1110-2-2504, Design of Sheet Pile Walls (Mar 94).

(7) EM 385-1-1, Safety Manual (Apr 81), Revised Oct 87.

b. Technical publications:

(1) American Concrete Institute, Building Code Requirements for Reinforced Concrete, (ACI 318-89).

(2) American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, Ninth Edition, 1989.

(3) American Welding Society, Structural Welding Code, Steel, (AWS-D 1.1-88).

(4) Concrete Reinforcing Steel Institute, CRSI Handbook, (1984).

c. Computer programs:

(1) "Pile Group Analysis (CPGA)," WES Program No. X0080.

(2) "CFRAME," WES Program No. X0030.

(3) "CWALSHT," WES Program No. X0031.

(4) "BEAMS," WES Program No. X0015.

9. Design Criteria.

a. General. The structural design presented herein complies with standard engineering practice and criteria set forth in Engineering Manuals, Engineering Technical Letters, and Engineering Regulations for civil works construction published by the Office, Chief of Engineers, and applicable technical publications. Structural calculations are included in Appendix B.

b. Material weights. The following material weights were used in the calculations.

Item:	PCF
Water	62.5
Concrete	150.0
Steel	490.0
Saturated Sand	122.0
Saturated Clay	110.0
Saturated Random Backfill	115.0
Riprap	132.0

c. Design stresses.

(1) Structural steel. The basic stresses for structural steel shall be in accordance with the American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, as modified by EM 1110-1-2101. EM 1110-1-2101 requires that AISC allowable stresses be reduced by 17%, as a basis for design. The structural steel shall be in accordance with ASTM A36.

(2) Welds. The allowable stresses for the design of welds shall be in accordance with the American Welding Society, Structural Welding Code, Steel, as modified by EM 1110-2-2101.

(3) Steel sheet piling. The basic stresses for steel sheet pile used in the cantilevered I-walls and temporary cofferdam shall be in accordance with EM 1110-2-2504. The steel sheet piling for permanent construction shall be in accordance with ASTM A328. The grade of steel sheet piling used for the temporary cofferdam system shall be as required for the selected cofferdam design. Allowable stresses for the cofferdam shall be increased due to the temporary nature of the structure.

(4) Reinforced concrete. The design of reinforced concrete shall be by strength design methods and criteria established in EM 1110-2-2104.

f'c	3,000 psi
Maximum flexural reinforcement	0.25 * balance ratio
Minimum flexural reinforcement	200 / fy

(5) Steel H-piling. The design stresses for steel H-piles shall be in accordance with EM 1110-2-2906. The steel H-piles shall be in accordance with ASTM A36. The allowable stresses for steel H-piles are summarized as follows:

axial compression or tension - lower region: 10.0 KSI

combined bending and axial compression - upper region:

$$\frac{fa}{Fa} \pm \frac{fbx}{Fb} \pm \frac{fby}{Fb} \leq 1.0$$

where:
fa = computed axial unit stress
Fa = (0.833)*(0.600)*fy = 18.0 KSI (ASTM A36)
fbx, fby = computed bending unit stress
Fb = (0.833)*(0.600)*fy = 20.0 KSI (ASTM A36; compact)

(6) Allowable overstress. The basic stresses noted above shall be modified as follows:

Loading condition	Allowable overstress
construction	16-2/3%
maintenance dewatering	33-1/3%
* usual conditions	0%
* unusual conditions	33-1/3%
* extreme conditions	50%

* See Loading Conditions for definition of usual, unusual, and extreme conditions for each monolith.

10. Loading Conditions.

a. General. The Standard Project Hurricane (SPH) level is El. 11.9 NGVD. For the I-wall, T-wall, and gated monoliths, usual loading conditions include a canal stage at the SPH level. Unusual loading conditions include a canal stage at the top of the protection, El. 13.9 NGVD. An extreme loading condition was used only for the gated monolith for the 1000 CFS pumps and is discussed below. For all hydraulic conditions, i.e. conditions including hydrostatic loads, two uplift conditions are used to account for the effectiveness of the sheet pile cut-off under the monoliths.

b. Gated monolith for 1000 CFS pumps. The emergency gates in this monolith will be closed only to prevent backflow in the event of pump failure during a hurricane or storm surge. Backflow through the pump discharge system cannot occur until the canal stage reaches El. 8.0 NGVD. In the usual and unusual loading conditions, the canal stage was 11.9 NGVD and 13.9 NGVD respectively. Gates were assumed to be closed when the canal stage reached El. 8.0 NGVD. In the extreme condition, the canal stage is 11.9 NGVD. Gates were assumed to be closed prematurely at canal stage El. 1.0 NGVD, the normal canal stage. For these load cases, all three pumps were assumed failed to maximize the monolith foundation loads. In the maintenance dewatering cases, a canal stage of El. 4.0 NGVD, the average annual high canal stage, was assumed. In the construction case, the structure was assumed to be completed but not yet submerged. The construction case was used only for foundation analysis.

Typical foundation design calculations for this monolith are shown on plates 13 and 14.

Structural and foundation designs are based on the following load cases:

(1) Usual conditions.

(a) Gate closed, canal SWL El. 11.9 NGVD, SWL inside discharge culverts El. 8.0 NGVD, storm wind load, impervious sheet pile cutoff.

(b) Gate closed, canal SWL El. 11.9 NGVD, SWL inside discharge culverts El. 8.0 NGVD, storm wind load, pervious sheet pile cutoff.

(2) Unusual conditions (*).

(a) Gate closed, canal SWL El. 13.9 NGVD, SWL inside discharge culverts El. 8.0 NGVD, storm wind load, impervious sheet pile cutoff.

(b) Gate closed, canal SWL El. 13.9 NGVD, SWL inside discharge culverts El. 8.0 NGVD, storm wind load, pervious sheet pile cutoff.

(3) Maintenance dewatering conditions.

(a) Dewatering stop logs installed, canal SWL El. 4.0 NGVD, SWL inside discharge culverts El. -11.0 NGVD, operating wind load, impervious sheet pile cutoff.

(b) Dewatering stop logs installed, canal SWL El. 4.0 NGVD, SWL inside discharge culverts El. -11.0 NGVD, operating wind load, pervious sheet pile cutoff.

(*) For the foundation analyses only, the total monolith loads were reduced by 25% and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-⅓% allowable overstress.

(4) Construction condition. No hydrostatic load, no wind load. This case considers the completed structural components prior to watering.

(5) Extreme condition. Gate closed, canal SWL El. 11.9 NGVD, SWL inside discharge culverts El. 1.0 NGVD, storm wind load, impervious sheet pile cutoff.

c. Gated discharge basin for 320 CFS pumps. The emergency gates in this monolith will be closed only to prevent backflow in the event of pump failure during a hurricane or storm surge. Backflow through the pump discharge system cannot occur until the canal stage reaches El. 4.57 NGVD. In the usual and unusual loading conditions, the canal stage was 11.9 NGVD and 13.9 NGVD respectively. Gates were assumed to be closed when the canal stage reached El. 3.57 NGVD. No extreme condition was used due to the small decrease in resisting pressures resulting from premature gate closure. Both pumps were assumed failed to maximize the monolith foundation loads.

In the maintenance dewatering cases, a canal stage of El. 4.0 NGVD, the average annual high canal stage, was assumed. In the construction case, the structure was assumed to be completed but not yet submerged. The construction case was used only for foundation analysis.

The gates will be offset from the face of the existing pumping house to allow sufficient space to mount the gate frames without partially blocking the existing discharge culvert mouths. The south side wall will be sloped, westward of the I-wall connection, from an elevation of 14.4 NGVD to an elevation of 0.0 NGVD.

Typical foundation design calculations for this monolith are shown on plate 15.

Structural and foundation designs are based on the following load cases:

(1) Usual conditions.

(a) Gate closed, canal SWL El. 11.9 NGVD, SWL inside discharge culverts El. 3.57 NGVD, storm wind load, impervious sheet pile cutoff.

(b) Gate closed, canal SWL El. 11.9 NGVD, SWL inside discharge culverts El. 3.57 NGVD, storm wind load, pervious sheet pile cutoff.

(2) Unusual conditions (*).

(a) Gate closed, canal SWL El. 13.9 NGVD, SWL inside discharge culverts El. 3.57 NGVD, storm wind load, impervious sheet pile cutoff.

(b) Gate closed, canal SWL El. 13.9 NGVD, SWL inside discharge culverts El. 3.57 NGVD, storm wind load, pervious sheet pile cutoff.

(3) Maintenance dewatering conditions.

(a) Dewatering stop logs installed, canal SWL El. 4.0 NGVD, SWL inside discharge culverts El. -8.5 NGVD, operating wind load, impervious sheet pile cutoff.

(b) Dewatering stop logs installed, canal SWL El. 4.0 NGVD, SWL inside discharge culverts El. -8.5 NGVD, operating wind load, pervious sheet pile cutoff.

(4) Construction condition. No hydrostatic load, no wind load. This case considers the completed structural components prior to watering.

(*) For the foundation analyses only, the total monolith loads were reduced by 25% and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-⅓% allowable overstress.

d. T-wall monoliths. In the usual and unusual loading conditions, the canal stage was 11.9 NGVD and 13.9 NGVD respectively. Rain water trapped between the new and old floodwalls was assumed to be immediately pumped into the canal, eliminating protected-side hydrostatic head. In the construction case, the structure was assumed to be completed but not yet submerged. The construction case was used only for foundation analysis.

Eliminating the protected-side hydrostatic head produced an unusually thick stem base. Typical foundation design calculations for these monoliths are shown on plates 11 and 12.

Structural and foundation designs are based on the following load cases:

(1) Usual conditions.

(a) Canal SWL El. 11.9 NGVD, storm wind load, impervious sheetpile cut-off.

(b) Canal SWL El. 11.9 NGVD, storm wind load, pervious sheetpile cut-off.

(2) Unusual conditions (*).

(a) Canal SWL El. 13.9 NGVD, storm wind load, impervious sheetpile cut-off.

(b) Canal SWL El. 13.9 NGVD, storm wind load, pervious sheetpile cut-off.

(*) For the foundation analyses only, the total monolith loads were reduced by 25% and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-⅓% allowable overstress.

(3) Construction conditions.

(a) No hydrostatic load, operating wind load from flood side.

(b) No hydrostatic load, no wind load, dead load only.

e. I-wall monoliths. In the usual and unusual loading conditions, the canal stage was 11.9 NGVD and 13.9 NGVD respectively. The monoliths were designed as cantilevered structures. The concrete cap top elevation of 14.4 NGVD was used to account for up to six inches of settlement with respect to the pile-founded structures, which have a top elevation of 13.9 NGVD. The steel sheet pile top elevations, the concrete cap bottom elevations, and the finished grade elevations were selected to match the adjoining I-wall monoliths at the north and south ends of the project. Flexible joints will be used at intersections with the north-end T-wall and the gated discharge basin to account for the differences in deflection under high canal stage loading. The concrete caps will conform to the standard tapered design with a 12 inch top stem thickness.

Typical foundation design calculations for these monoliths are shown on plates 28-30.

Structural and foundation designs are based on the following load cases:

(1) Usual conditions. Q-Case (soil FS = 1.5) with SWL El. 11.9 NGVD.

(2) Unusual conditions. Q-Case (soil FS = 1.0) with SWL El. 13.9 NGVD.

f. Operating floor. The operating floor sections for the gated monoliths for 1000 CFS pumps and the gated discharge basin monolith must sustain maximum installation and operating loads.

Structural designs are based on the following load cases:

(1) Usual conditions.

(a) Maximum installation loads which develop during placement or removal of the deck sections.

(b) Maximum operating loads which develop during normal gate operation.

(2) Extreme condition: Maximum operating loads which develop when gate jams during opening or closing.

11. Structural Design. Structural design calculations are shown in Appendix B. Each of the proposed structures was designed as follows:

a. Gated monolith for 1000 CFS pumps. Structural components of the gated monoliths were designed as follows:

(1) Longitudinal wall. The longitudinal wall was designed to transfer load horizontally to the piers. The wall was assumed to have fixed ends and span between the piers. Out-of-plane loading was used to determine required flexural and shear reinforcement.

(2) Piers. The piers were designed to transfer load vertically to the base slab. The piers were assumed fixed at the longitudinal wall and the base slab. Both in-plane (Case I) and out-of-plane (Case V) loading were used to determine required flexural and shear reinforcement at the base of the pier. Hydrostatic loading was used to determine required gate guide (Case I) and stoplog (Case V) contact areas to avoid local concrete bearing overstress.

(3) Slab. The slabs were designed to transfer load horizontally to the piles. The slab was assumed fixed to the pier(s) and pinned to the pile heads. Out-of-plane loading was used to determine required flexural and shear reinforcement in both the transverse and longitudinal directions. Program CFRAME was used to analyze strips of the slab at critical locations.

b. Gated discharge basin for 320 CFS pumps. Structural components of the gated monolith were designed as follows:

(1) Longitudinal wall. The longitudinal wall was designed to transfer load horizontally to the piers. The wall was assumed to have fixed ends and span between the piers. Out-of-plane loading was used to determine required flexural and shear reinforcement. This design is similar to the wall design for the gated monolith for the 1000 CFS pumps.

(2) Piers. The piers were designed to transfer load vertically to the base slab. The piers were assumed fixed at the longitudinal wall and the base slab. Both in-plane (Case I) and out-of-plane (Case V) loading were used to determine required flexural and shear reinforcement at the base of the pier. Hydrostatic loading was used to determine required gate guide (Case I) and stoplog (Case V) contact areas to avoid local concrete bearing overstress. This design is similar to the pier design for the gated monolith for the 1000 CFS pumps.

(3) Slab. The slabs were designed to transfer load horizontally to the piles. The slab was assumed fixed to the pier(s) and pinned to the pile heads. Out-of-plane loading was used to determine required flexural and shear reinforcement in both the transverse and longitudinal directions. Program CFRAME was used to analyze strips of the slab at critical locations.

(4) Side walls. The side walls were designed to transfer load vertically to the base slab. The side walls were assumed fixed at the base slab. Out-of-plane loading was used to determine required flexural and shear reinforcement.

c. T-wall monoliths. Structural components of the T-wall monoliths was designed as follows:

(1) Stem. The stem was designed to transfer load vertically to the base slab. The stem were assumed fixed at the base slab. Out-of-plane loading was used to determine required flexural and shear reinforcement.

(2) Slab. The slabs were designed to transfer load horizontally to the piles. The slab was assumed fixed to the stem and pinned to the pile heads. Out-of-plane loading was used to determine required flexural and shear reinforcement in both the transverse and longitudinal directions. Program CFRAME was used to analyze strips of the slab at critical locations.

d. I-wall monoliths. Structural components of the I-wall monoliths were designed as follows:

(1) Concrete cap. The concrete cap was designed to transfer load vertically to the steel sheet piling. Out-of-plane loading was used to determine required flexural and shear reinforcement in vertical direction.

(2) Sheet piling. The steel sheet piling was designed to transfer load vertically into the soil. The sheet piling was analyzed as a cantilever with program BEAMS, based on net pressure diagrams produced with program NODWAL. The required tip penetration was based on a soil strength safety factor of 1.5, while the required sheet pile section was based on a soil strength safety factor of 1.0 and a steel strength safety factor of 2.0.

e. Dewatering bulkheads. The dewatering bulkheads, i.e. stoplogs, were designed to transfer load horizontally to the gated monolith piers. Out-of-plane loading was used to determine required flexural and shear reinforcement in both the transverse and longitudinal directions.

f. Temporary cofferdam. The cofferdam design was designed to retain water to SWL El. 5.75 NGVD, and transfer load into the soil. Out-of-plane loading was used to determine the required steel sheet pile size, steel wide flange wale size, and the battered anchor pile penetration. The cofferdam was analyzed based on five foot anchor pile spacing. Program CWALSHT was used to develop the net pressure diagram on the sheet piling. Program CFRAME was then used to analyze the sheet piling and anchor pile system.

12. Foundation Design. Pile foundation designs were based on three dimensional pile group analyses using the program Pile Group Analysis (CPGA). For pile layouts, spacing and batter were based on minimizing the base slab width and avoiding interference with the existing timber piles. Pile heads were assumed to be pinned and pile caps were assumed to be rigid. Q-case soil conditions were assumed for all analyses. All horizontal load transfer was assumed to occur within the top soil stratum (sand). See pile capacity curve plates.

For each of the pile founded structures, i.e. the gated monoliths and the T-wall monoliths, pile foundation designs assume use of a pile test and a soil strength factor of safety of 2.0. Pile cap movement was limited to 0.5 inch so that no load is imparted to the existing structure, i.e. the structures were designed to withstand the entire load system. Stabilization slab loads were not included in foundation design.

For the gated monolith designs, water trapped in the culverts in the usual and unusual load cases provided enough lateral and overturning resistance to the monolith to significantly reduce the required pile lengths and limit the lateral monolith movement to acceptable displacements. In each of these load cases, the foundation analyses proved to be sensitive to pile batter. Pile batters for the compression piles were limited by the existing timber piles supporting the pumping station slab. Also, eccentric loading caused higher corner pile loads for the T-wall monoliths and the gated monolith for the 1000 CFS pumps due to the rigid cap assumption of CPGA. For simplicity, one final pile length, sufficient for all piles in these monoliths, was selected. A second pile length was selected for the gated discharge basin monolith.

13. Cathodic Protection and Corrosion Control. Cathodic protection and corrosion control for steel sheet piling, steel gates, corner plates and all other ferrous metal components of the fronting protection plan shall be provided.

METHOD OF CONSTRUCTION

14. General. The Contractor shall construct the fronting protection from a floating plant within the canal to minimize interference with station operations. See paragraph 6h for a complete description of the temporary cofferdam system to be used. Steel sheet piling for the first cofferdam shall saddle the overhead ten-foot diameter siphon and fifty-inch discharge line. Installation of the steel foundation piles under these obstructions shall be accomplished by splicing sections of hydraulically jacked or vibratory hammer driven piles. The construction easement shall include the vacant NOSWB property at the north end of the pumping station and the vacant NOSWB property on the west side of the canal, across from the pumping station. All electrical relocations will be coordinated with NOSWB. All piping and miscellaneous steel modifications will be designed by NOSWB and included in the construction contract.

ACCESS ROADS

15. Vehicular Access. Vehicular access to the project site is available via many public roads. Roads adjacent to the site are Prentiss Avenue and Warrington Drive, from the east side, and Pratt Drive from the west side. Access into the canal may be gained from the canal bridges at Filmore Avenue and Robert E. Lee Blvd.

RELOCATIONS

16. General. Under the authorizing law, local interests are responsible for the accomplishment of "... all necessary alterations and relocations to roads, railroads, pipelines, cables, wharves, drainage structures and other facilities made necessary by the construction work ...".

17. Utility Relocations. Where relocated utility lines cross steel sheet piling, steel sleeves will be installed to allow the utility lines to pass through the floodwall. Water tight seals will be placed around the lines. Temporary bypass lines may be required. All utility relocations will be included in the construction contract.

a. Electric feeder lines. The following electric feeder lines will be affected by project construction:

(1) FL-340, at the south end of the station, will be temporarily relocated land side of its existing position. No splicing for additional length is anticipated.

(2) FL-422, at the south end of the station, will be temporarily relocated canal side of its existing position. A steel sleeve and water tight seal will be installed for cable penetration of new I-wall. No splicing for additional length is anticipated.

(3) FL-400, at the north end of the station, will be permanently relocated. A steel sleeve and water tight seal will be installed for penetrating the existing I-wall approximately twenty feet south of the existing penetration location. No splicing for additional length is anticipated.

b. Telephone cable. The existing South Central Bell telephone cable rack housing 18-5" telephone cables at the south end of the station will not be relocated. A steel sleeve and water tight seal will be installed for cable penetration of new I-wall. The existing buried South Central Bell telephone cables are to be abandoned, and removed as required for installation of the new I-wall.

c. Power pole. The existing power pole will be relocated by New Orleans Public Service Inc. (NOPSI), to a position acceptable to the USACE.

d. Piping. The NOSWB will design all pumping station piping relocations or modifications. The anticipated relocations or modifications minimally include the 12' vacuum discharge lines for the 1000 CFS pumps, the 24" siphon line for the 1000 CFS siphon, and the 14" vacuum suction line penetrating the north discharge basin wall. All modifications will be included in the fronting protection construction contract.

e. Miscellaneous steel. The NOSWB will design all pumping station miscellaneous steel relocations or modifications. The anticipated relocations or modifications minimally include the south end crane support and pipe/equipment support steel adjacent to the north side of the discharge basin. All modifications will be included in the fronting protection construction contract.

MECHANICAL

18. General. The design of the mechanical system for the fronting protection will include provisions for eight gate assemblies and two permanent dewatering pumps. The design is based on the use of equipment and material that are available as standard industry products. In the selection of equipment, consideration will be given to ease of operation, reliability, and ease of maintenance.

19. Sluice Gate Operators. The sluice gates will be individually closed only to prevent backflow when a pump is disabled or a power outage occurs during hurricane or flood conditions. Operation will be by local and remote push button control and indicating lights. Operation of the eight sluice gates will be by individual electric actuators that will require approximately eleven minutes per gate to fully close or open. Starting the actuators one at a time and allowing three gates to operate simultaneously will provide a total operating time, from full open to full closed, of approximately 30 minutes for all eight gates. Each actuator will be furnished with a bracket for mounting a portable air motor to operate the gates in the event of a power outage. Two portable air motors will be provided.

Limit switches in the actuator's control panel will control the gate's open and closed positions, while torque limiting switches, also in the control panel, will automatically stop the motor if the gate were to encounter an obstruction during its upward or downward motion. Additionally, fused disconnects in the station's electrical control panel will automatically interrupt power to the motor in order to prevent it from developing its locked rotor torque.

20. Permanent Dewatering Pumps. Two portable submersible pumps will be installed at the base of the flood wall, sized to drain the accumulation of a 4 inches rainfall on the station over a period of four hours, i.e., 50 gpm at a total head of 26 ft. The pumps will be supported by discharge connectors permanently anchored to the concrete floor and guided by guide bars on the concrete flood wall to allow their removal for service. The pump's cantilevered weight provides an automatic seal between its discharge flange and the discharge connector's flange without the need of bolts or threaded connections. The 2 inch diameter stainless steel discharge line will be permanently anchored to the flood wall and will discharge over the wall into the outfall channel. Actuation of the pumps will be automatic through float level controllers.

ELECTRICAL DESIGN

21. General. The design of the electrical system for the eight gate motors and controllers will include provisions for power and control. The design is based on criteria provided by the NOSWB, concerning space conduit routing and power source availability, and on the use of equipment and material that are available as standard products of the electrical industry. The NOSWB will provide a minimum 150 amp., 25 hz. panel service in order to allow the sequential start up and simultaneous operation of three gates at a time. In the selection of materials and equipment, consideration will be given to ease of operation, reliability, and ease of maintenance. The Standards of the National Electrical Manufacturers Association (NEMA), the Institute of Electrical and Electronic Engineers (IEEE), and the American National Standards Institute (ANSI) will be used as guides in the selection of electrical equipment. The design of circuits and conduit systems will conform to the 1993 National Electrical Code and the National Electrical Safety Code.

22. Power Source. NOSWB will provide a 480 V motor feeder circuit from their Low Voltage Motor Control Center located in the main building.

23. Power Distribution.

a. General. Power will be supplied from a fused disconnect switch located in a low voltage motor control center in the main building.

b. Loads. Eight 25 hz., 480V, 3-phase motors, one for each gate, will be supplied from the motor feeder circuit. Reversing combination starters will be located adjacent to each gate. Starters will allow local operation of each gate as well as remote operation from a new NOSWB control console. Two 60 hz., 200V, 3-phase, 2 Hp de-watering pumps will be supplied from an existing 208Y/120 V panel located in the Low Voltage Motor Control Center.

c. Voltage drop requirements. Conductors will be sized to prevent the voltage drop from exceeding three percent at the farthest utilization point on each circuit.

24. Conduit and Boxes.

a. Conduit. All wiring will be installed in rigid metal conduit except that motors and other electrical equipment subject to vibration, will be connected with liquid-tight flexible metal conduit.

b. Pull and junction boxes. All pull boxes and junction boxes will be of cast metal of sufficient thickness or provided with bosses to accommodate the required threads for the conduit connections of size specified.

HYDROLOGY AND HYDRAULICS

25. General. Design Memorandum No. 19A General Design London Avenue Outfall Canal Orleans Parish presents the hydraulic analysis performed for the London Avenue Outfall Canal to determine the required levee/floodwall height for hurricane protection.

26. Hydraulic Design.

a. Discussions have been held with the NOSWB on the recommended plan for fronting protection at Pumping Station No. 4. The main concern coming out of these discussions is the effect of the proposed fronting protection on canal flows, particularly during low and normal lake stages. It is anticipated that the fronting protection will induce additional head losses and possibly create unfavorable current patterns.

b. The NOSWB has initiated a model study of the canal to address effects of projects they have planned unrelated to the fronting protection at Pumping Station No. 4. Agreement has been reached to include an analysis of the proposed fronting protection plan in the model study to determine head losses and changes to current patterns. It is not anticipated that results of the model study will require changes to the fronting protection plan. Remedial measures can be taken to compensate for additional head losses and adverse current patterns. It is believed, though depending on results of the model study, that the resulting head losses will be offset by the removal of the Benefit Street bridge and three pedestrian bridges. Widening the west side of the canal in the vicinity of the pumping station could also be performed if required.

c. Standard hydraulic design procedures were used to check for head loss associated with the proposed gate arrangements for the sluice gates for the 1000 CFS pumps and for the 2-320 CFS pumps. These procedures are outlined in Chapter 6 of King & Brater "Hand Book of Hydraulics." Because of the proposed configuration of the gates and their placement flush against the concrete sides of the discharge tubes, the only additional head that the pumps will see will be as a result of the entrance loss due to piers between gates and friction losses associated with the an almost completely turbulent pipe flow through the sluice. There will be a sudden expansion loss at the downstream end of the sluice, but this loss would have occurred without the addition of the gates. However, the sum of the losses computed for the proposed configuration was less than 0.1 ft and presents no problem.

GEOLOGY

27. General.

a. Scope. The geology presented herein is based on the geology from Design Memorandum No. 19A General Design London Avenue Outfall Canal (January 1989), which was based on regional, local surface and subsurface information. Additional subsurface information supplemented the data from GDM No. 19A. It is intended to present a general project overview of the pertinent geologic data and interpretation.

b. Physiography and topography. The project site is located within the Central Gulf Coastal Plain region on the flanks of the Mississippi River Deltaic Plain and normal to the Lake Pontchartrain shoreline in northern Orleans Parish. Pronounced physiographic features of the area are lakes, shorelines, canals, an abandoned Mississippi River delta, the Mississippi River, beach ridges, marshes and swamps. Elevations in the vicinity vary from -15.0 feet NGVD in Lake Pontchartrain to +20.0 feet NGVD along the crown of the mainline Mississippi River levees.

c. Surface investigation. Aerial photographs, topographic maps, and geologic maps were used in conjunction with published literature to define the geologic setting of the project area.

d. Subsurface investigation. One 1⅞-inch I.D. general type boring and one 5-inch undisturbed boring were drilled by Corps of Engineers personnel for this project. In addition a total of seven 3-inch and 5-inch A-E contract undisturbed borings were reviewed for additional geologic classification by Corps of Engineer personnel. All borings are included on the geologic profiles (plate 19) in order to present the most geologically complete interpretation. The A-E contract boring symbols were modified to accommodate the Unified Soil Classification system. All borings encountered artificial fill and Holocene soils. Those borings exceeding 70 feet generally encountered the Pleistocene horizon. The boring data, used in conjunction with other available data, was the primary source for site specific geologic foundation interpretations.

e. Geophysical investigation. No geophysical methods were used at the project site. Present refractive methods would not have delineated the various Holocene environments.

28. Regional Geology. Reference Design Memorandum No. 19A General Design, London Avenue Outfall Canal for information on regional geology.

29. Site Geology.

a. Site location and description. The project is confined to northern Orleans Parish and that portion of the levee that parallels the London Avenue Outfall Canal at Pumping Station No. 4. This represents approximately 300 feet of levee improvement. The project alignment is nearly normal to the regional geologic strike and traverses Holocene surficial marsh and subsurface beach, and marine deposits. A review of the geologic profile on plate 24 details geologic structure parallel to levee centerline. Subsurface elevations at the top of Pleistocene average -60 feet, but vary from approximately -55 to -70 feet. Depth to top of the Pleistocene increases southward from the lakeshore to Pumping Station No. 3.

Historically, the site stratigraphic sequence indicates a period of aerially exposed Pleistocene prior to an early Holocene marine transgression. Evidence of a gulfwater transgression and the subsequent development of the Pontchartrain Embayment is present as a locally extensive basal bay-sound deposit. The clayey bay-sound deposit averages 20 feet in thickness and provides parenting material for the overlying Pine Island Beach trend. Estimated ages of the beach and bay-sound deposits are respectively 5,000 and 7,000 years.

Isolation of the embayment by the eastward prograding Cocodrie Delta (4,600 to 3,500 years before present) marked the end of marine conditions. Cocodrie aged deposits appear to be absent or obscured in the immediate area. This is possibly a result of two factors: (1) the deltaic material was eroded after abandonment and (2) the remaining material closely resembles the overlying lacustrine and further testing would be necessary to differentiate.

The later prograding St. Bernard Delta, 2,800-1,700 years ago, represented the last major period of active deltaic sedimentation within the area. The surficial marsh deposit was deposited during recent time. West of the project, marsh type deposits are found within the confines of Lake Pontchartrain. This may be evidence of an expanding lake resulting from shoreline retreat.

The surficial marsh veneer, 5 to 15 feet thick throughout most of the London Avenue Canal, represents the last stage of sedimentation in the area. Marsh type sediments are a result of annual Mississippi River overbank flooding and subsequent deposition of clay and silt size particles landward of the natural levees.

A review of borings in the vicinity of the artificial levee indicates that the additional overburden acts as a surcharge, in some instances consolidating the underlying marsh deposit to less than half the original thickness. Along the centerline of the artificial levee, the additional loading of soil has, to a lesser extent, similarly affected the underlying lacustrine deposit.

b. Detailed Holocene environmental descriptions.

(1) Bay-sound deposits are fine to coarse grain sediments bottoming bays and sounds. Average thicknesses are 15 feet in the project area. Reworking of the bottom portion by burrowing marine organisms produces a mottled appearance and inclusions of materials that are distinct from the surrounding sediment. Colors are typically light gray to gray.

(2) Beach deposits are typically fine sands with large quantities of shells and shell fragments. The sands, generally well sorted with few clay lenses, are well suited for founding projects. Subsidence due to soil compaction is relatively minimal. The base elevation of the deposit remains a relatively constant -50 feet NGVD. This deposit is the remnant Pine Island Beach trend.

(3) The marsh deposits are highly compressible organic soils that typically cover 95 percent of area. They grade vertically downward from peat to organic clays and silts. Generally, soil moistures exceed 100 percent, color varies from light gray to black, and consistences vary from very soft to medium.

c. Detailed Pleistocene soil descriptions. The Pleistocene soils are a result of both deltaic and marine deposition. They represent both the regressive and transgressive phases and associated environments of an earlier Mississippi River deltaic system. The soils are, therefore, similar to the overlying Holocene. However, due to dessication, Pleistocene deposits are distinguished by a decrease in moisture contents, a stiffening of consistencies, a decrease in sampling penetration rates, an increase in oxidized sediments, and the presence of calcareous concretions.

d. Foundation conditions. Representative geologic site conditions are displayed on cross-sections shown on plate 19. The massive beach deposit has greatly influenced the stratigraphic geometry of the area.

e. Future investigations. Subsurface field investigations have been completed, and no future investigations are anticipated.

30. Conclusion. Current geologic information indicates generally favorable foundation conditions with regard to future construction. Further addition of fill may result in increased settlement rates, due to marsh soil compaction. Differential settlement may result in areas where organic contents are extremely high and relatively thick. Should future construction in the immediate project vicinity require dewatering local settlement may occur due to oxidation of organics and consolidation of sediment.

FOUNDATION INVESTIGATION AND DESIGN

31. General. This section includes the soils investigations and foundation design for Pumping Station No. 4 fronting protection plan. The plan consists of I-walls in levees, and pile supported structures.

32. Field Exploration. One continuous undisturbed 5-inch diameter soil boring, 10-LUG, was made in the project area at the protected side levee toe. The individual log is shown on plate 20. The location of the undisturbed boring is shown on plate 2. Boring 10-LUGA an augerboring was taken in the existing levee and is shown with boring 10-LUG in profile on plate 21. Two borings number 56 and 57 taken by an A-E for the Orleans Levee Board were used in conjunction with the COE borings in the foundation design. Boring 57 was made with a 5-inch diameter Shelby Tube sampling barrel. The locations of borings taken by the A-E are shown on plate 2 and the boring logs are contained in Appendix A.

33. Laboratory Tests.

a. COE. All samples obtained from borings 10-LUG and 10-LUGA were visually classified. Water content determinations were made on all cohesive soil samples. Unconfined Compression (UC) Shear Tests, and Atterbergs were made on selected samples of cohesive soils. Water content determination and (UC) test results are shown adjacent to the logs on the boring profile presented on plate 21. Unconsolidated-Undrained (Q), Consolidated-Undrained (R) and Consolidated Drained shear tests were made on representative soil samples obtained from the undisturbed samples. These tests are summarized on the boring log shown on plate 20. The individual shear strength sheets are shown in Appendix A.

b. A-E. Laboratory tests consisting of natural water content, unit weight, and either Unconfined Compression (UC), or Unconsolidated-Undrained (Q) one point shear tests were performed by the A-E on samples obtained from the A-E borings. Liquid and plastic limit tests were made on selected samples. Laboratory test results are shown in Appendix A. UC and one point Q tests in silts and sands were not plotted on the design shear strength profiles.

c. Design shear strength. Design shear strength parameters are shown on plate 22.

34. Design Problems.

a. Cofferdam required to construct the structure in the dry. Elevation of cofferdam, restricted working area and access.

b. Dewatering and hydrostatic pressure relief required to construct the structure in the dry.

c. Bearing pile lengths and subgrade reaction data for the fronting protection sluice gate structure and T-walls.

35. Construction Dewatering and Hydrostatic Pressure Relief. To build the structure in the dry and insure stability of the structure excavation during construction, hydrostatic pressure relief will be provided in the pervious layers in the excavation area. Temporary piezometers will be installed in the pervious layers to monitor the pressure during the dewatering and pressure relief period. The method of lowering the ground water is to be left to the construction contractor with performance specifications being prepared on an "end result" basis. The specifications will allow the use of wells, sumps, pumps, etc., as well as well points. The theoretical radius of influence of 700 ft. estimated for dewatering the excavation will include residential homes. If the buildings are on piles no monitoring will be necessary. The contractor's dewatering proposal will provide for surface control points to measure settlements adjacent to existing structures. Final approval of the proposed system will include provisions for installation of recharge wells if the radius of influence, magnitude of drawdown and soil stratification indicates that settlement will occur. The dewatering system presented on plate 23 is for cost estimating purposes for use in evaluating the adequacy of the contractor's hydrostatic pressure relief system.

36. Underseepage and Hydrostatic Pressure Relief.

a. Underseepage. A steel sheet pile cutoff wall will be used beneath the T-wall and sluice gate structure to provide protection against detrimental seepage. The location and penetration depth of the sheet pile cutoff wall are shown on plate 6. Analyses were performed by the Harr Method. The calculations are shown on plate 32.

b. Hydrostatic pressure relief. A piezometric headline based on the ground surface and past piezometric readings (GDM No. 19A London Ave Outfall Canal) was used in the stability and uplift analyses.

37. Pile Foundations.

a. Ultimate compression and tension pile capacities versus tip elevations were developed for steel HP 14X73 piles plate 24, and timber piles plate 25. Overburden stresses were limited to D/B=15 in the sands. Soil design parameters are shown on plate 22. Values used in the pile capacity calculations are shown in Tables 1 and 2. The results of design pile loads versus tip elevations for cost estimating purposes are based on applying factors-of-safety shown in Table 3. The timber pile capacity curves are used to estimate the pile capacity of the existing timber piles below the discharge structure.

b. During construction, test piles will be driven and load tested for the fronting protection. The results of the pile load tests will be used to determine the length of the service piles. The pile test will be conducted on the north side of the pumping station on the protected side of the flood protection.

c. The settlement of the sluice gate structure is estimated to be 0.2 ft. from consolidation below the pile tips in the first pleistocene horizon.

d. Subgrade moduli curves for estimating lateral resistance of the soil beneath the discharge basin, sluice gate structure and pile supported T-wall were developed and are shown on plates 24 and 25.

TABLE 1
TIMBER PILES

	Q-CASE					S-CASE				
	ø	K_c	K_t	N_c	N_q	ø	K_c	K_t	N_c	N_q
Clay	0°	1.0	0.7	9	0	23°	1.0	0.7	0	10
Sand	30°	1.0	0.7	0	22.5	30°	1.0	0.7	0	22.5

TABLE 2
STEEL H-PILES

	Q-CASE					S-CASE				
	ø	K_c	K_t	N_c	N_q	ø	K_c	K_t	N_c	N_q
Clay	0°	1.0	0.7	9	0	23°	1.0	0.7	0	10
Silt	15°	1.0	0.7	12.9	4.4	30°	1.0	0.7	0	22.5
Sand	30°	1.0	0.7	0	22.5	30°	1.0	0.7	0	22.5

TABLE 3
RECOMMENDED FACTORS OF SAFETY
FOR PILE CAPACITY CURVES

	WITH PILE LOAD TEST	W/O PILE LOAD TEST
Q-CASE:	2.0	3.0
S-CASE:	1.5 TOTAL LOAD 2.0 DEAD LOAD ONLY	2.25 TOTAL LOAD 3.0 DEAD LOAD ONLY

38. Shear Stability. Stability was determined by the LMVD Method of Planes analysis for a minimum factor of safety of 1.3 with respect to the design shear strength. The borings used to develop a design shear strength profile are shown on plate 21 and Appendix A. Plates 26 and 27 show the protected and floodside stability analyses for the I-wall in levee between Sub B/L Sta 0+00 to 0+76.89 and Sub B/L Sta 2+56.08 to Sub B/L Sta 3+00.58.

39. I-Wall. The required penetration of the steel sheet piling below ground surface was determined by the Method of Planes using "Q" shear strengths. The Q-Case design strengths are based on data shown on plate 22. The factors of safety were applied to the design shear strengths as follows: ϕ developed = arctan ϕ (tan available/factor-of-safety) and c/factor-of-safety. Using the resulting shear strengths, net lateral soil and water pressure diagrams were developed for movement toward each side of the sheet pile. With these pressure distributions, the summation of horizontal forces was equated to 0 for various tip penetrations and the overturning moments about the tip of the sheet pile were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of moments was equal to zero. Following is sheet pile wall design criteria for hurricane protection levees:

TIP PENETRATIONS

Q-CASE

F.S. = 1.5 With water to SWL
F.S. = 1.0 With water to SWL + 2ft freeboard

DEFLECTIONS

F.S. = 1.0 With water to SWL

BENDING MOMENTS

F.S. = 1.0 With water to SWL

If the penetration to head ratio is less than about 3:1, it is increased to 3:1. The SWL is used to calculate head for penetration to head ratio.

Plates 28 and 29 show I-wall analyses for Sub B/L Sta. 0+00 to Sub B/L Sta. 0+76.89 and Sub B/L Sta. 2+73.08 to Sub B/L Sta. 3+00.58 which are similar to the sections of the I-Walls now under construction in the contract London Ave Outfall Canal Mirabeau Ave. to Leon C. Simon Blvd., East Bank. Plate 30 shows a bulkhead analysis for Sub B/L Sta. 2+47.39 to Sub B/L Sta. 2+73.08 which ties into the T-wall at Sub B/L Sta. 2+47.39.

Construction cofferdam analysis on plate 31 shows an anchor wall analysis for a sheet pile wall braced with steel H-piles. The top of the cofferdam is at El. 5.7 which corresponds to an El. 4.0 stage in Lake Pontchartrain.

40. T-Wall. A deep seated analysis utilizing a 1.3 factor of safety incorporated into the soil properties was performed for various potential failure surfaces beneath the T-wall. The analysis is shown on plate 32. The summation of horizontal driving and resisting forces results in a value that is positive at the base and negative as the elevation of the failure surface is lowered. Since the net driving forces are less than the net at-rest force the structure is assumed to be stable and all loads (vertical and horizontal) must be developed in pile capacity below the base.

41. Levee Settlement. No settlement is expected since the levees will either remain at the same grade or be degraded.

Based on service and environment conditions, a water-cement ratio of 0.58 will not be exceeded for durability requirements. The slump will range from 1 to 4 inches.

(2) Environmental conditions. The concrete will not be subjected to any critical environmental or functional conditions.

b. Cementitious materials investigation.

(1) Cement.

(a) Special requirements. Because of the nature of local aggregates, low alkali cement must be used. False set requirements will be necessary if an on-site batch plant is used, however a local ready mix plant will likely be chosen by the Contractor.

(b) Availability. Cement meeting Type I or II requirements of ASTM C 150 in addition to the above special requirements is locally available from Citadel Cement, LaFarge Co., Dundee Holnam Cement Co., Louisiana Industries, and others.

(c) Type and justification. Because of the availability of Type II cement at no additional cost and lower heat of hydration, Type II cement will be specified.

(d) Testing requirements. Testing requirements of CW-03301, paragraph 3.1.2.3 will be imposed in the specifications in lieu of paragraph 5.1.2.

(2) Pozzolan. Fly ash meeting the requirements of ASTM C 618, Types C or F, including the optional chemical and physical requirements 1A and 2A, respectively, will be allowed. The percentage of fly ash in the Contractor's furnished mix design will be limited to not greater than 35 percent of absolute volume. Its recommended use is based on potential cost savings. Also using fly ash could potentially reduce heat of hydration and permeability, and improve sulfate resistance. Type C fly ash obtained from Bayou Ash was satisfactorily used on the Old River Control Auxiliary Structure and is currently being satisfactorily used in the production of articulated concrete mats at St. Francisville, LA. Bayou Ash is located near New Roads, LA, approximately 120 miles from New Orleans, LA.

c. Aggregate investigation.

(1) Sand and gravel. The sources listed in Table 4 are a few of the area companies on our pretested list that seem capable of furnishing sand and gravel for the project.

Test reports can be found in TM 6-370 and Old River Control, LA, Auxiliary Structure Sources of Construction Materials, DM No. 14 dated 3 Oct 80. Transportation of aggregates would probably be by truck, except for Lambert Gravel which has also indicated that barging from their source is possible.

TABLE 4
SAND AND GRAVEL SOURCE LIST

Company Name	Nearest Town (LA)	Project to pit Distance (miles)	Pit Location Lat (deg)	Pit Location Long (deg)	TM 6-370 Vol/ Area	TM 6-370 Index Number
Lambert Gravel	Bains	130	30	91	4A/9A	1
La. Industries	Enon	70	30	90	4A/9A	9
Rebel Sand & Gravel	Watson	102	30	90	3A/7A	16
Standard Gravel	Enon	70	30	90	4A/9A	28
T.L. James & Co.	Pearl River	45	30	89	4A/9A	11

d. Concrete batch plant and truck mixer investigation.

(1) On-site batch plant. The largest single concrete placement appears to be the discharge basin slab which its volume is approximately 100 cubic yards. The concrete batch plant needs to have a capacity of at least 50 cubic yards per hour in order to prevent cold joints during placement.

(2) Off-site batch plant. Ready mix concrete meeting the requirements of this project and produced from batch plants meeting the guidelines of Cast-in-place Structural Concrete (CW-03301) can be obtained from the sources listed in Table 5:

TABLE 5
CONCRETE SOURCE LIST

Company Name	Distance to Project (miles)	Plant Capacity (CY/HR)	Plant Type	Number of Truck Mixers	Cooling Method
LA Indus. (Plant 4) (Euphrosinte St.)	5	100	Semi	23	ice
LaFarge (Airline Hwy)	20	180	Auto	52	ice or chilled water
Ditta Carlo (S. Peter St.)	10	120	Auto	36	ice

e. Thermal considerations. The largest single concrete placement will be the 2.5-foot thick base slab of the discharge basin. Its volume is approximately 100 cubic yards. The placing temperature of the base slab concrete will not be allowed to exceed 85 degrees F, while for other elements the maximum will be 90 degrees F.

ENVIRONMENTAL

43. General. The London Avenue Outfall Canal is a man-made canal approximately 4.0 miles in length, with an average bottom and top width of 100 to 160, respectively. Pumping Station No. 3 lies at the head of the canal near Broad Street. Pumping Station No. 4 is near Prentiss Avenue. The canal is paralleled by earthen levees topped with floodwalls or floodwalls alone from Pumping Station No. 3 to Leon C. Simon Boulevard on the east and to Robert E. Lee Boulevard on the West. From these two boulevards to Lakefront Drive there is an earthen levee on both sides of the canal.

44. Existing Conditions. Vegetation on the flood side of the protection consists of scrub-shrub habitat with baccharis, giant ragweed, mulberry and elderberry predominating. In some areas of the canal narrow bands of roseau cane are located along the waters edge. The levee is kept mowed and covered with annual and perennial grasses and weeds.

Because of the high incidence of human disturbance, the area provides marginal habitat for wildlife. Some use of shrubs and trees by squirrels and songbirds occurs. Terns and seagulls are sometimes seen feeding along the exposed canal bottom.

Water quality in the canal is generally poor and normally exceeds criteria for propagation of fish and wildlife. The canal provides minimal value as habitat for fishery resources.

Some recreational opportunities exist in the vicinity. The levee is used by joggers, walkers, bird watchers and fishermen. Fishing is primarily limited to the lakefront area. Picnicking and field sports utilize the adjacent levee near University of New Orleans.

Esthetics is generally poor due to the poorly maintained areas around the pumping stations and the condition of the floodwall. Positive esthetic conditions exist on both sides of the canal north of Robert E. Lee Boulevard. The open grassy areas west of the canal has scattered oaks and oleanders and are well maintained.

No cultural resources or endangered species are recorded in the vicinity of the proposed work.

Noise levels in the area are within the range expected for residential areas and somewhat lower in the park-like areas. The residents in the project area support the project and will not be displaced by its construction.

45. Environmental Effects. There would be minimal short term displacement of tree-dwelling animals and songbirds. This impact would be minor due to the small number of trees impacted. All recreational uses in the work areas would cease during construction. Following construction the impacted areas would again support the same kinds of recreational activities in pre-project conditions. Esthetics would be degraded in some areas due to the reduced viewing resulting from the increased levee height. This is minor impact when compared to the esthetics provided by the redesigned and graphically textured floodwall. There would be no impact on significant cultural resources or endangered species. The ambient noise level would be increased during construction with some residences close to the construction site experiencing noise levels that could interfere with sleeping, conversation and some recreational activities. These levels will occur only for the

period of construction and will be limited to the daylight hours. There will be some temporary disruption in normal traffic patterns during construction, but will be limited again to daylight hours. No displacement of residences will be necessary.

46. Environmental Compliance. The final Environmental Impact Statement (EIS), for Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection Project, was filed with the President's Council on Environmental Quality on 17 January 1975. A Final Supplement to this EIS was filed with the Environmental Protection Agency (EPA) in December of 1984. The Final Supplement assessed the impacts associated with increased levee height for a high level plan of protection.

The impacts of providing protection along the outfall canals were not addressed in the original EIS or the subsequent supplement. However, an Environmental Assessment (EA), addressing the impacts associated with providing hurricane induced flood protection, for the London Avenue Canal, was prepared on 7 October 1988. Based on this EA, a determination was made that the hurricane protection provided along this canal would not have a significant impact upon the human environment. A Finding of No Significant Impact (FONSI) was signed 27 October 1988. This completes the environmental compliance for construction of this feature.

ESTIMATE OF COST

47. General. Based on October 1994 price levels, the estimated first cost for constructing the fronting protection at Pumping Station No. 4 is $3,754,000. A cost of $80,000 is for relocations and $2,988,000 for the levees and floodwalls feature. Engineering and Design and Construction Management are estimated to be $388,000 and $298,000. Table 6 presents the itemized first cost for the fronting protection at Pumping Station No. 4.

TABLE 6
LONDON AVENUE OUTFALL CANAL
PUMPING STATION NO. 4
FRONTING PROTECTION
(Aug 94 Price Levels)

Cost Acct. No.	Description	Estimated Quantity	Unit	Unit Price $	Estimated Amount $
02---	RELOCATIONS:				
023--	CEMETERIES, UTILITIES, STRUCTURES, Construction Activities				
0232-	Utilities:				
	Structural				
	Misc. Structural Steel (*Coord. with NOSWB)		LS	1,500.00	1,500.00
	Mechanical				
	Piping & Fittings Coordinated by NOSWB)				
	24" Siphon Line	50	LF	290.00	14,500.00
	12" Vacuum Discharge Line	200	LF	200.00	40,000.00
	Misc. Piping and Supports		LS	5,500.00	5,500.00
	Electrical (*Coordinated by NOSWB)				
	Feeder Line - 340	75	LF	35.00	2,625.00
	Feeder Line - 422	75	LF	35.00	2,625.00
	Feeder Line - 400	75	LF	35.00	2,625.00
02---	**SUBTOTAL: Relocations**				**69,375.00**
	Contingencies				**10,625.00**
	TOTAL: Relocations				**80,000.00**
11---	FLOODWALLS AND LEVEES:				
110A	Mobilization, Demobilization and Preparatory Work:	LUMP SUM	LS	200,000.00	200,000.00
110B-	Care and Diversion of Water:				
110BQ	Dewatering System:				
	Site Work:				
	Selective Demolition:				
	Remove Canal Liner/Soil (Wet Conditions)	75	CY	100.00	7,500.00
	Temporary Cofferdam:				
	Steel Sheet Piling, PZ-40, Driving	27,250	SF	4.00	109,000.00
	Steel Sheet Piling, PZ-40, Pulling	27,250	SF	3.00	81,750.00
	HP 14X73 Steel Piles, Driving	8,050	LF	31.50	253,575
	HP 14X73 Steel Piles, Pulling	8,050	LF	10.00	80,500.00
	Steel Wide Flange Wales	690	LF	28.00	19,320.00
	Backfill:				
	Random Backfill	50	CY	6.00	300.00
	Mechanical Equipment:				
	Unwatering Pumps	LUMP SUM	LS	2,000.00	2,000.00
	Well-point system		LS	122,000.00	122,000.00
1102-	Floodwalls:				
1102B	Site Work:				
	Clearing and Grubbing	0.5	AC	2,00.00	1,000.00
	Selective Demolition:				
	Remove Existing Basin Concrete & Soil	170	CY	100.00	17,000.00
	Remove Existing Basin Cut-off Sheet Piling	500	FT	2.00	1,000.00
	Remove Existing Canal Liner and Soil	830	CY	100.00	83,000.00
	Remove Existing Fencing	30	FT	2.00	60.00

TABLE 6 (Cont'd)
LONDON AVENUE OUTFALL CANAL
PUMPING STATION NO. 4
FRONTING PROTECTION
(Aug 94 Price Levels)

Cost Acct. No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
				$	$
	Sheet Piling:				
	Steel Sheet Piling, PZ-22, Driving	7,200	SF	12.50	90,000.00
	Steel Sheet Piling, PSA-23 Tee, Driving	150	SF	13.00	1,950.00
	Piles:				
	HP14X73 Steel Piles, Driving	9,440	LF	31.50	297,360.00
	Backfill:				
	Canal Liner Repair - Concrete	110	CY	250.00	27,500.00
	Canal Liner Repair - Soil	270	CY	25.00	6,750.00
	Clay Backfill	630	CY	12.00	7,560.00
1102C	Concrete:				
	Concrete, in-place including Reinforcement:				
	Stab. Slab, Unreinforced	46	CY	70.00	3,220.00
	Monoliths, Reinforced	1,050	CY	330.00	346,500.00
	Waterstops, L-type	100	LF	30.00	3,000.00
	Waterstops, 3-bulb	625	LF	10.00	6,250.00
	Joint Filler	3,000	SF	2.00	6,000.00
1102N	Special Construction:				
	Corrosion Protection Systems:				
	Standard Iron Body Ferrule		LS	0.00	**
	Bond Cables		LS	0.00	**
**Included in Concrete					
1102M	Metals:				
	Embedded Metals:	10,000	LB	1.25	12,500.00
	Grating and Handrail, Aluminum	3,000	LB	30.00	90,000.00
	Walkway and Stair, Steel	2,600	LB	1.25	3,250.00
1102ME	Mechanical Equipment:				
	Gates and Valves				
	Backflow Prevention Gates with Operators:				
	Sluice, Manufactured:				
	120"X132" Gate Assemblies	6	EA	97,000.00	582,000.00
	96"X84" Gate Assemblies	2	EA	67,000.00	134,000.00
	Unwatering Pumps:				
	50 GPM Submersible Pump	2	EA	1,000.00	2,000.00
110R-	Associated General Items:				
110RR	Electrical:				
	Hoisting Assembly Electrical Equip.		LS	0.00	***
	Sluice Gate Electronic Controls		LS	0.00	***
	Lighting System		LS	0.00	***
***Included in Sluice Gate					
11---	**SUBTOTAL: Floodwalls and Levees**				**2,597,845.00**
	Contingencies				**390,155.00**
	TOTAL: Floodwalls and Levees				**2,988,000.00**
30---	ENGINEERING AND DESIGN (E&D)				
30G--	Feature Design Memorandum (FDM)				
30H--	Plans and Specifications (P&S)				
30HB-	Final Plans and Specifications				266,330.00
30HD-	Bidability, Constructability, and Operability Review				13,030.00

TABLE 6 (Cont'd)
LONDON AVENUE OUTFALL CANAL
PUMPING STATION NO. 4
FRONTING PROTECTION
(Aug 94 Price Levels)

Cost Acct. No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
				$	$
30K--	Engineering During Constriction (EDC)				48,310.00
30P--	Cost Engineering				19,800.00
30Q--	Construction and Supply Contract Award Activities 5,000.00				
30---		SUBTOTAL: Engineering and Design (E&D)			352,470.00
		Contingencies			35,530.00
		TOTAL: Engineering and Design (E&D)			388,000.00
31---	CONSTRUCTION MANAGEMENT	LUMP SUM	LS	259,200	259,200
31---		SUBTOTAL: Construction Management			259,200.00
		Contingencies			38,800.00
		TOTAL: Construction Management			298,000.00
		SUBTOTAL:		2,278,890.00	
		CONTINGENCY:		475,110.00	
		TOTAL:		3,754,000.00	

48. Schedule for Design and Construction. The sequence for design and construction is shown in Table 7.

TABLE 7
SCHEDULE FOR DESIGN AND CONSTRUCTION

	DESIGN		CONSTRUCTION		
ACTIVITY	START	COMPLETE	ADVER.	AWARD	COMPLETE
P&S	Jan 95	Apr 96	Jun 96	Aug 96	Dec 97
CONSTRUCTION CONTRACT			Nov 91	Jan 92	Jan 95

49. Comparison of Estimates. The current Project Cost Estimate (LMV Form 17/PB-3) $3,230,000 effective 1 Oct 94 is for the relocations and the levees floodwalls features. The current estimate of $2,501,000 for these features represents a decrease of $729,000 when compared to the LMJ Form 17/PB-3 estimate. This reduction in cost is primarily due to a refinement of the designs from a survey scope to DM scope.

50. Federal and Non-Federal Cost Breakdown. The breakdown of Federal and non-Federal costs needed to construct the Fronting Protection at Pumping Station No. 4 described in Supplement No. 1 to GDM 19A is shown in Table 8 below:

TABLE 8
FEDERAL AND NON-FEDERAL COST BREAKDOWN
OCT 94 PRICE LEVELS

Item	Federal	Non-Federal	Total
Relocations, Fronting Protection & Levees	$2,627,800	$1,126,200	$3,754,000

OPERATIONS AND MAINTENANCE

51. General. All operations and maintenance (O&M) costs for this project will be Federal responsibility. The estimated O&M cost are shown in Table 9 below:

TABLE 9
OPERATIONS AND MAINTENANCE

Item	Annual Cost [1]
Sluice Gate Maintenance	$2,600
Gated Monolith Maintenance	500
I/T Wall Maintenance	200
Subtotal	$3,300
Contingency	700
TOTAL	$4,000

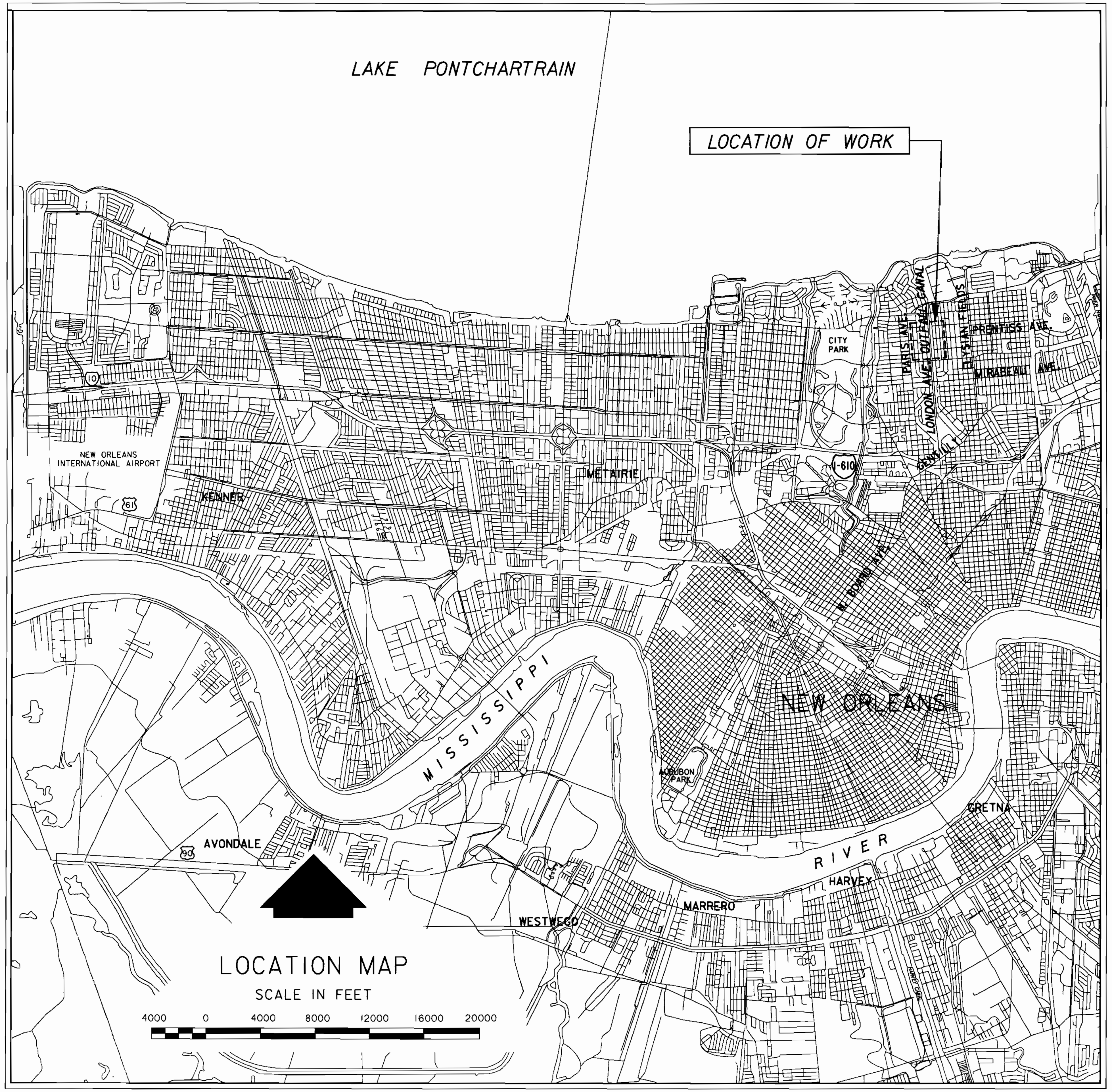
[1] The above annual cost estimates do not include replacement costs or increases due to inflation.

52. Funds Required by Fiscal Year. To maintain the schedule for design and construction for the Fronting Protection at Pumping Station No. 4 as shown in Table 7, funds will be required by fiscal year as shown in Table 10 below:

TABLE 10
TOTAL FEDERAL AND NON-FEDERAL FUNDING BY FISCAL YEAR

FY 95	$151,000
FY 96	$398,000
FY 97	$2,563,000
FY 98	$642,000

53. Recommendation. The plan of improvement recommended herein calls for construction of gated monoliths in front of the existing discharge culverts of Pumping Station No. 4 incorporating the use of I-walls and T-walls. The plan of improvement presented in this supplemental design memorandum is to sufficient detail to proceed to plans and specifications. Approval of this supplemental design memorandum is recommended.



NOTE:
DRAWINGS IN THIS FOLIO HAVE BEEN REDUCED ONE HALF THE ORIGINAL SCALE

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

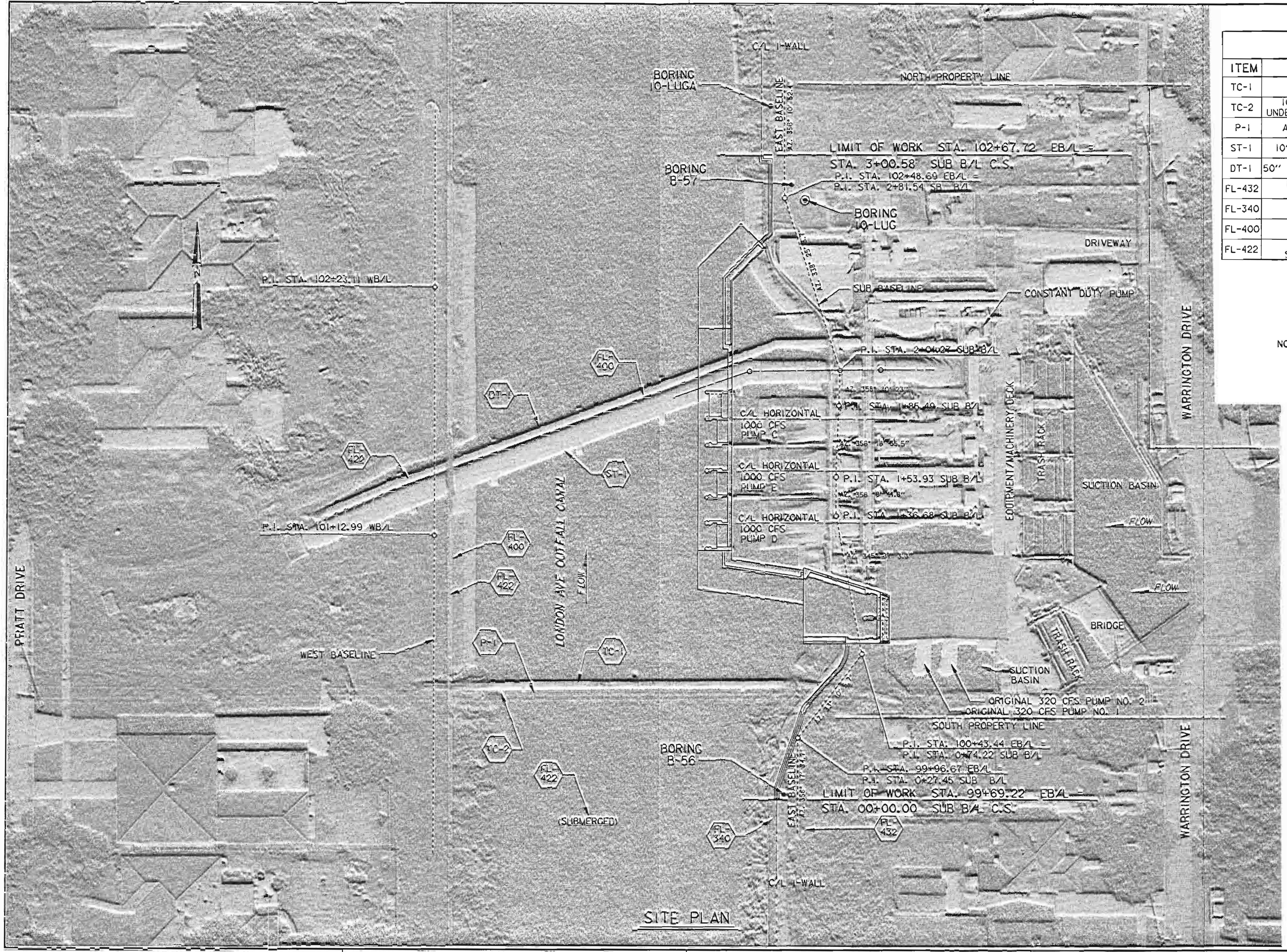
LOCATION AND VICINITY MAP

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

C•A•D

DESIGNED BY:	M.W.W.	PLOT SCALE: 4800	PLOT DATE: 9 JAN 95	CADD FILE: 40511N01.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 1



SITE PLAN

EXISTING FACILITIES

ITEM	DESCRIPTION	STA. WEST	STA. EAST	OWNER	
TC-1	18-5" TELEPHONE CONDUITS	100 + 49.61	100 + 18.61	SOUTH CENTRAL BELL	
TC-2	10-5" WIRE ARMORED UNDERGROUND TELE-CABLES	100 + 41.27	100 + 14.28	SOUTH CENTRAL BELL	•
P-1	AERIAL POWER LINES	100 + 46.20	100 + 11.37	N.O.P.S.I.	• •
ST-1	10' STEEL SIPHON TUBE	101 + 38.90	101 + 71.23	N.O.S.&W.B.	• • •
DT-1	50" STEEL DISCHARGE TUBE	101 + 47.29	101 + 84.41	N.O.S.&W.B.	• • •
FL-432	PRIMARY 25 CYCLE POWER CABLE	———	85 + 30.00 TO 101 + 63.00	N.O.S.&W.B.	• • •
FL-340	PRIMARY 25 CYCLE POWER CABLE	———	85 + 50.38 TO 100 + 13.79	N.O.S.&W.B.	• • •
FL-400	PRIMARY 25 CYCLE POWER CABLE	85 + 80.50 TO 101 + 47.29	———	N.O.S.&W.B.	• • •
FL-422	POWER CABLE SEWER STA. NO. 22	99 + 99.23 TO 101 + 38.90	99 + 88.47 TO 100 + 13.79	N.O.S.&W.B.	• • •

• TO BE ABANDONED
• • NEW ORLEANS PUBLIC SERVICE, INC
• • • NEW ORLEANS SEWERAGE AND WATER BOARD

NOTES:

SOIL BORINGS:

◉ UNDISTURBED TYPE
• GENERAL TYPE

SEE PLATE 24 FOR 10-LUG UNDISTURBED BORING LOG.

SEE PLATE 25 FOR 10-LUGA GENERAL BORING LOG.

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

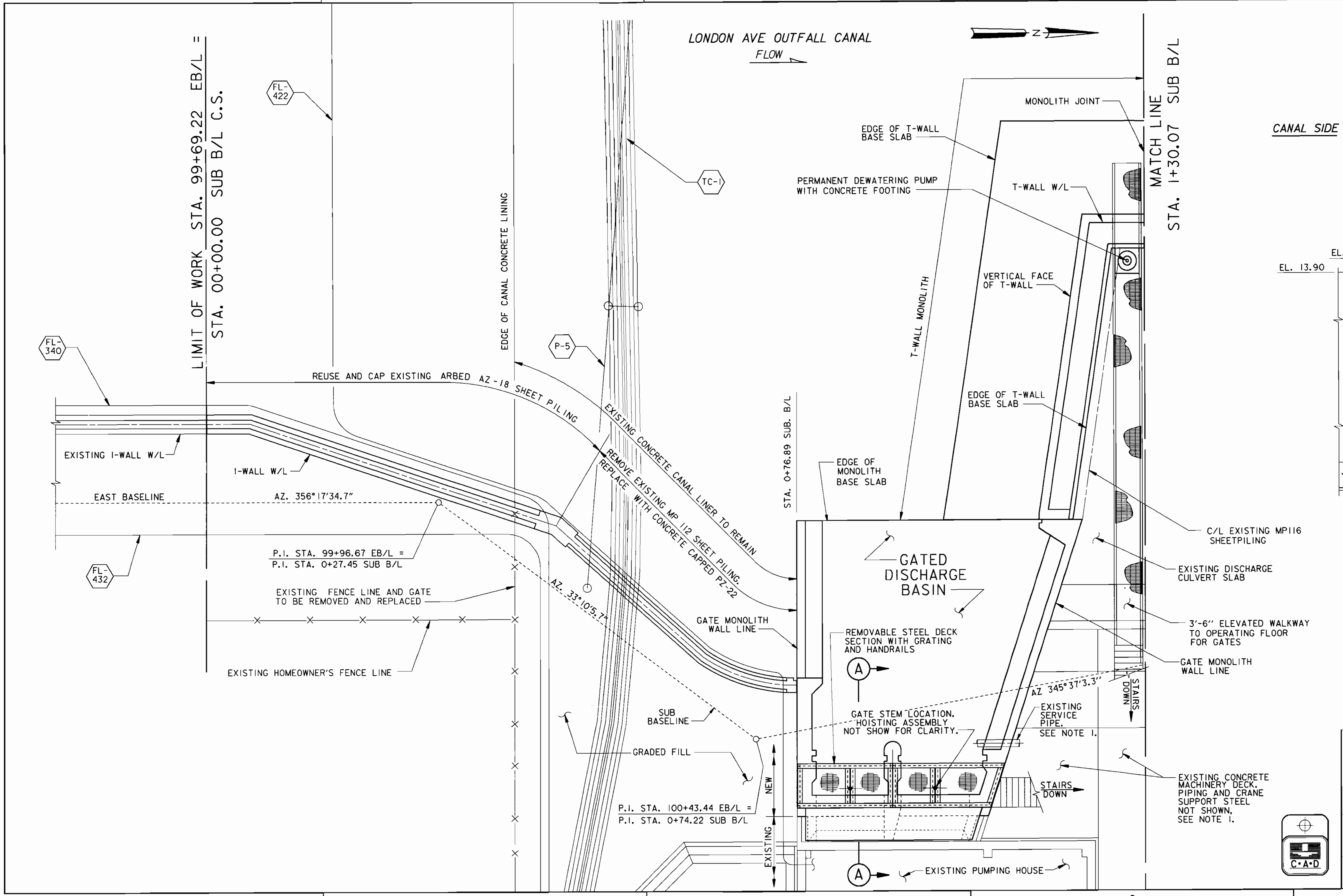
SITE PLAN

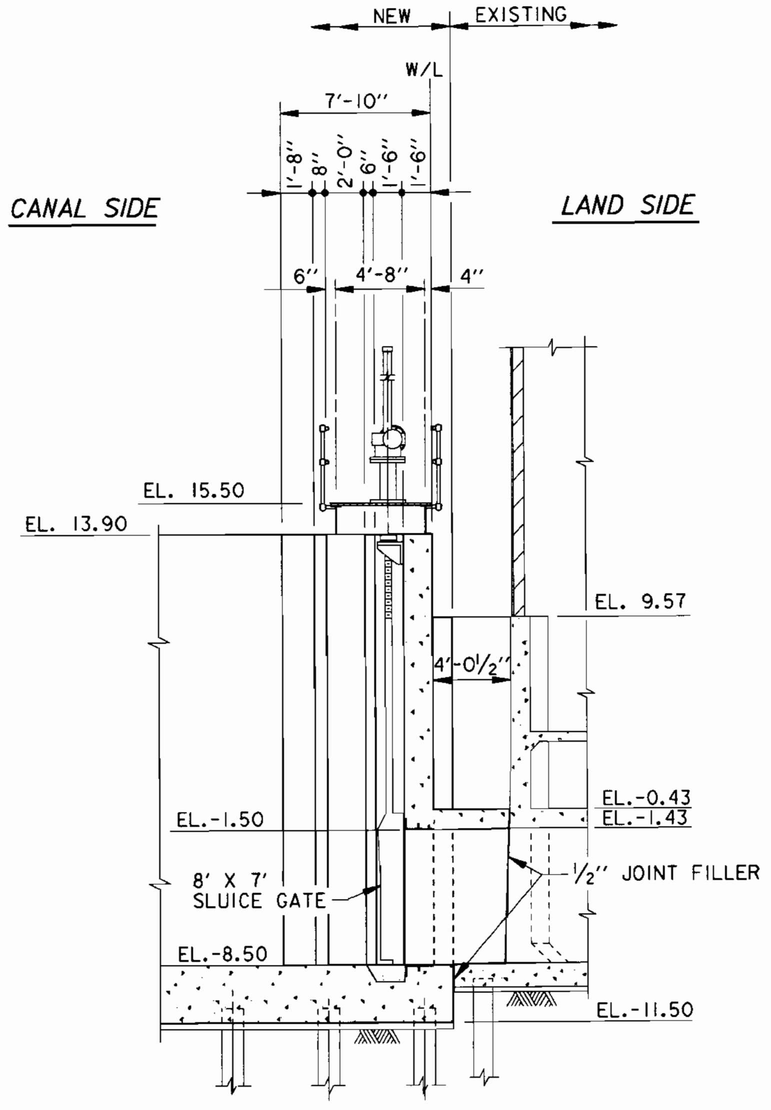
C·A·D

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W.
DRAWN BY: L.V.M./V.A.C.
CHECKED BY: W.O.B.

PLOT SCALE:	PLOT DATE:	CADD FILE: 40511POT.DGN
240	9 JAN 95	FILE NO. H-2-40511
DATE: JAN 1995		





SECTION A

SCALE: 1" = 5'

NOTES:

1. N.O.S.&W.B. TO DESIGN ALL PIPING AND STRUCTRUAL STEEL RELOCATIONS.

SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

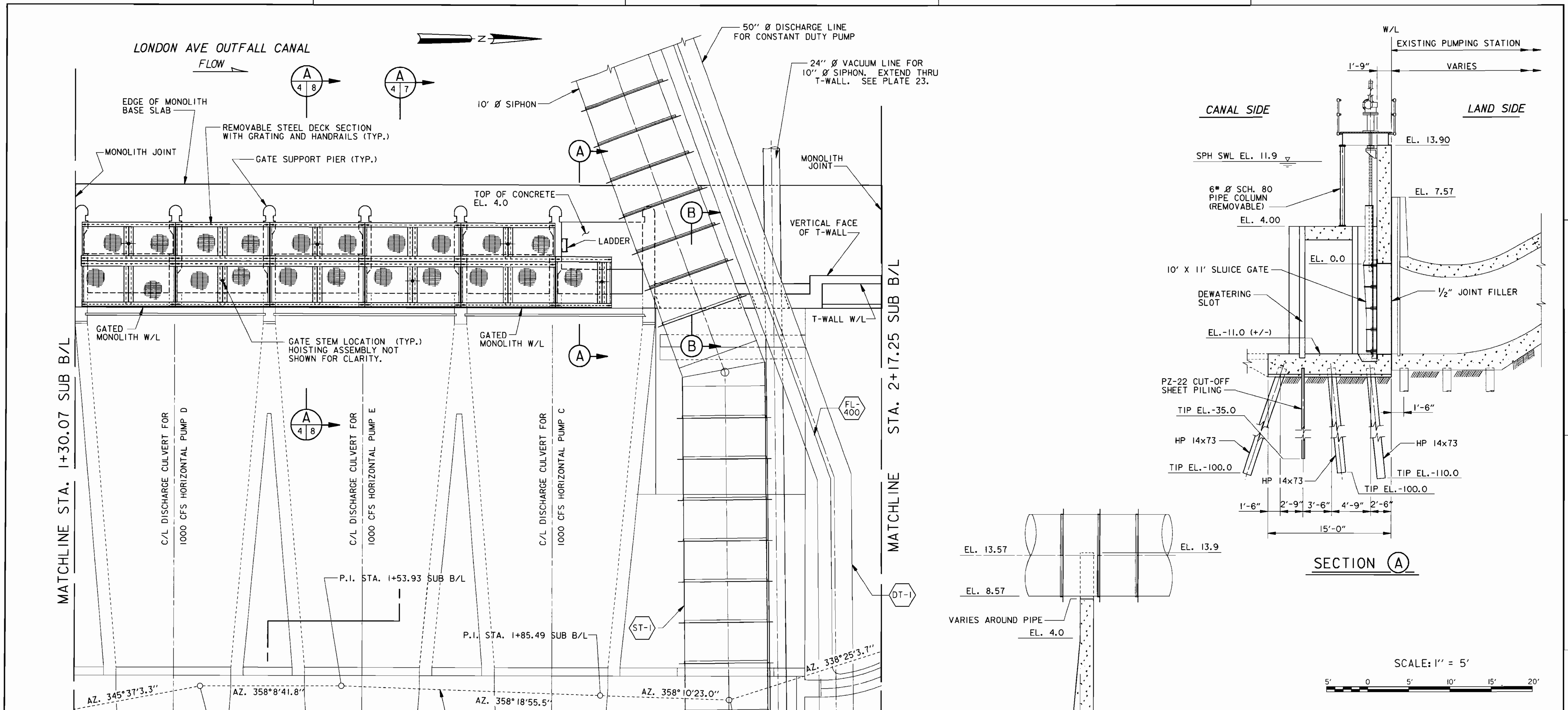
PLAN STA. 0+00 SUB B/L
TO STA. 1+30.07 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 60	PLOT DATE: 9 JAN 95	CADD FILE: 40511N25.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

C·A·D

PLATE 3

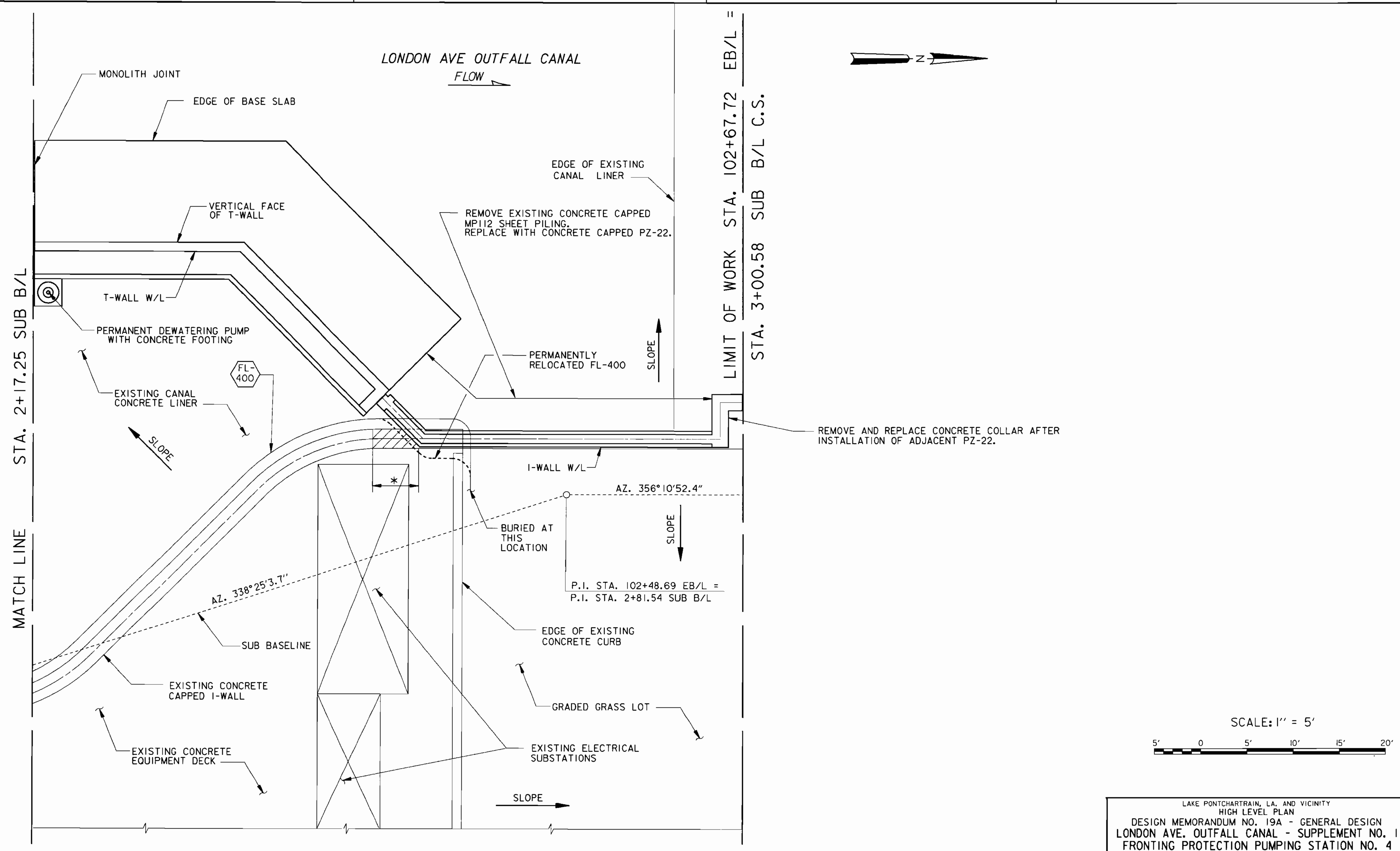


LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH
PLAN STA. 1+30.07 SUB B/L
TO STA. 2+17.25 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 60	PLOT DATE: 9 JAN 95	CADD FILE: 40511N26.DGN
DRAWN BY:	V.A.C.	DATE: JAN 1995		FILE NO. H-2-40511
CHECKED BY:	W.O.B.			

PLATE 4



* REMOVE EXISTING CONCRETE CAPPED FLOODWALL TO FACILITATE CONSTRUCTION OF NEW FLOODWALL AND PROVIDE ACCESS TO DEWATERING PUMP. (APPROX. 5')

PLAN STA. 2+17.25 SUB B/L
TO STA. 3+00.58 SUB B/L

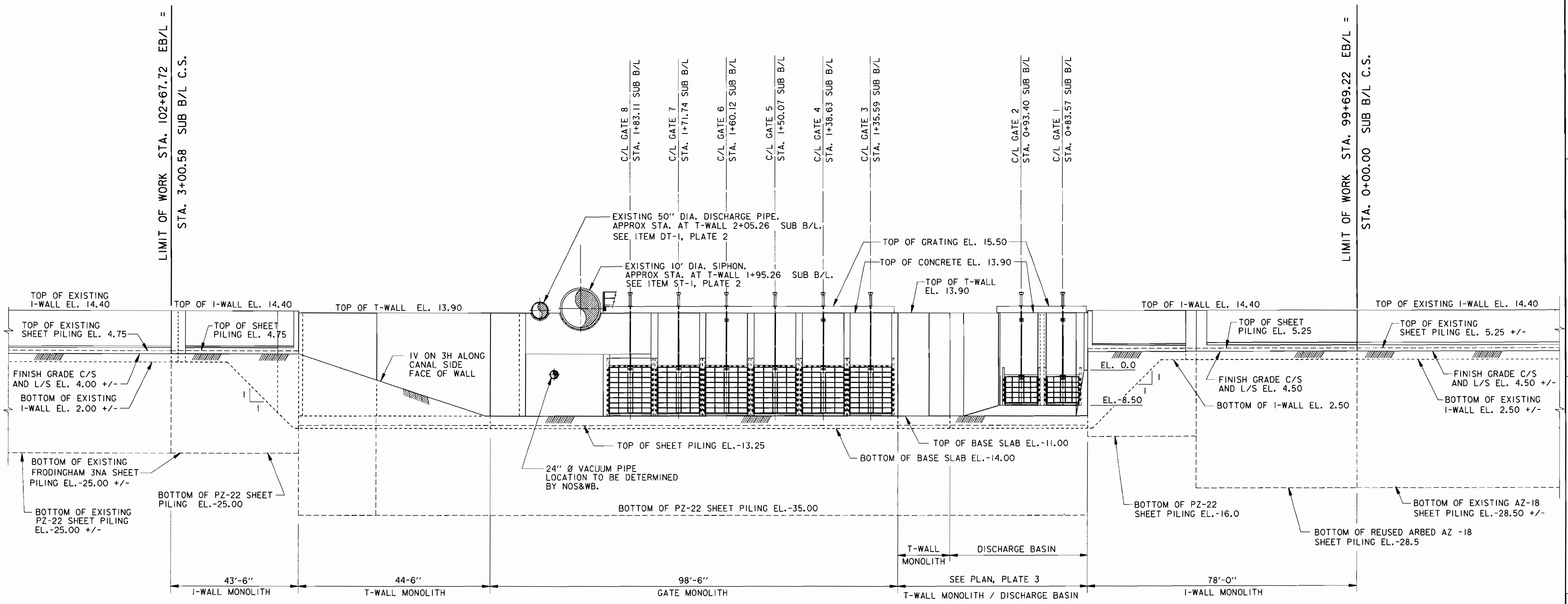
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

PLAN STA. 2+17.25 SUB B/L
TO STA. 3+00.58 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W.	PLOT SCALE: 60	PLOT DATE: 9 JAN 95	CADD FILE: 40511N27.DGN	
DRAWN BY: V.A.C.			FILE NO. H-2-40511	
CHECKED BY: W.O.B.	DATE: JAN 1995			

PLATE 5



NOTES:

1. STEEL H PILES NOT SHOWN FOR CLARITY.
2. MONOLITH LENGTHS ARE WALL LINE LENGTHS.
3. EXISTING PUMPING STATION NOT SHOWN FOR CLARITY.
4. GATE 1 AND 2: 96" X 84" CAST IRON SLUICE GATE
   GATE 3 TO 8: 120" X 132" CAST IRON SLUICE GATE

CANAL SIDE ELEVATION

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

C•A•D

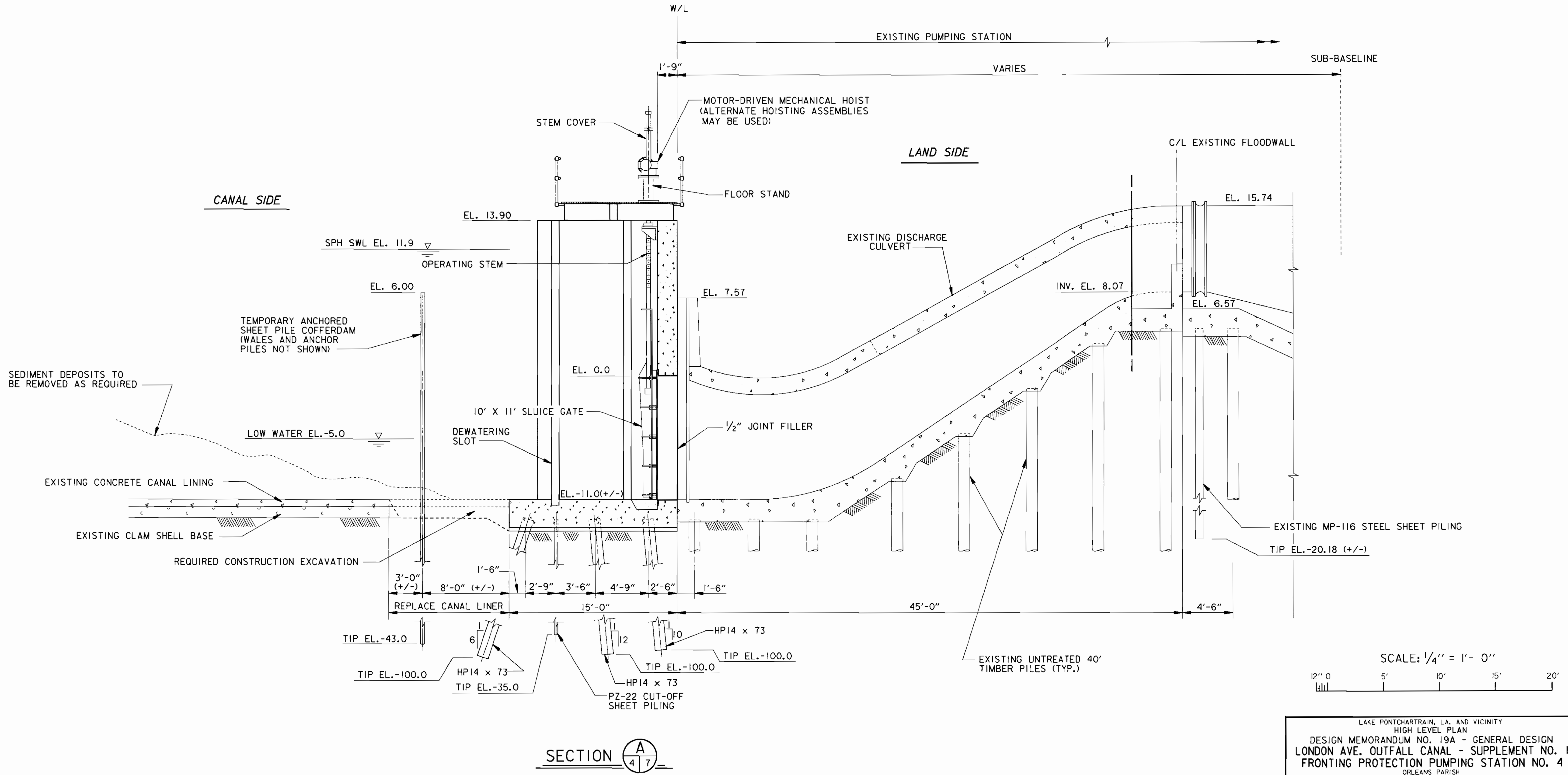
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

CANAL SIDE ELEVATION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 120	PLOT DATE: 9 JAN 95	CADD FILE: 40511P05.DGN
DRAWN BY:	L.V.M.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 6



SECTION A 4 7

NOTE:

1. EXISTING PUMPING STATION FACILITIES INCLUDING EQUIPMENT, MACHINERY, PIPING, UTILITY LINES, AND CRANE NOT SHOWN FOR CLARITY.

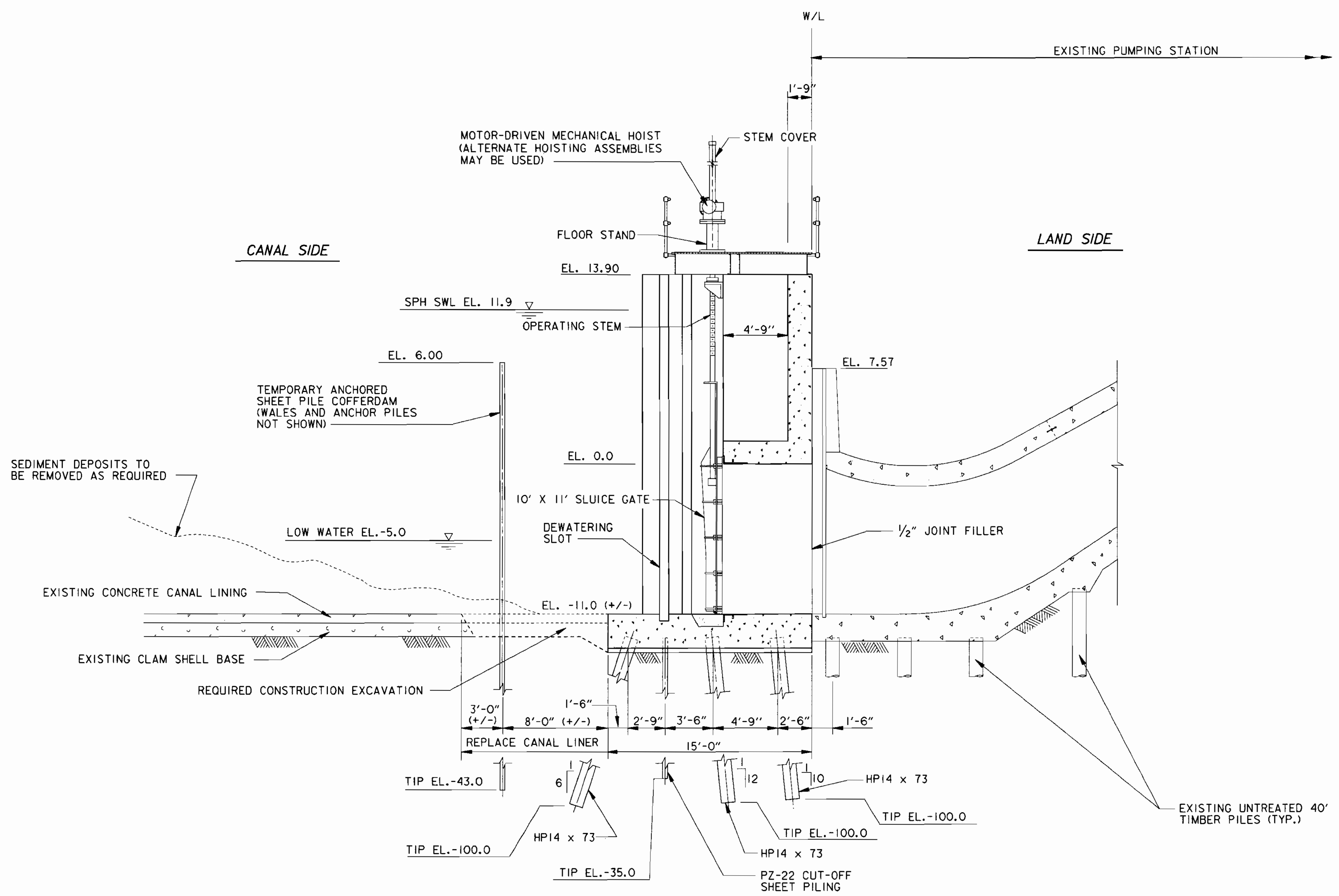
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

SECTION
THROUGH FRONTING PROTECTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W.	PLOT SCALE: 48	PLOT DATE: 9 JAN 95	CADD FILE: 40511N09.DGN
DRAWN BY: P.J.S./V.A.C.			FILE NO. H-2-40511
CHECKED BY: W.O.B.	DATE: JAN 1995		

PLATE 7



SECTION A 4 8

NOTE:

1. EXISTING PUMPING STATION FACILITIES INCLUDING EQUIPMENT, MACHINERY, PIPING, UTILITY LINES, AND CRANE NOT SHOWN FOR CLARITY.

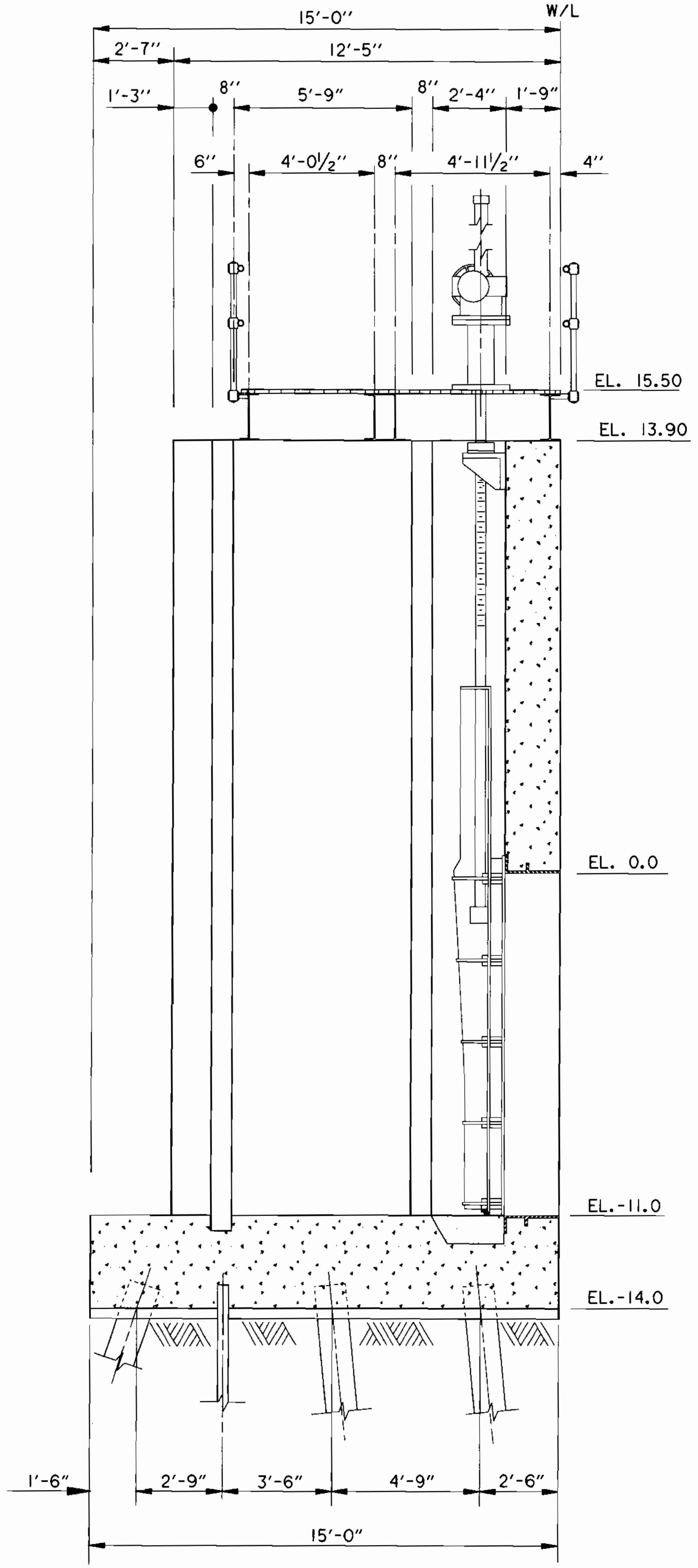
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

SECTION THROUGH FRONTING PROTECTION

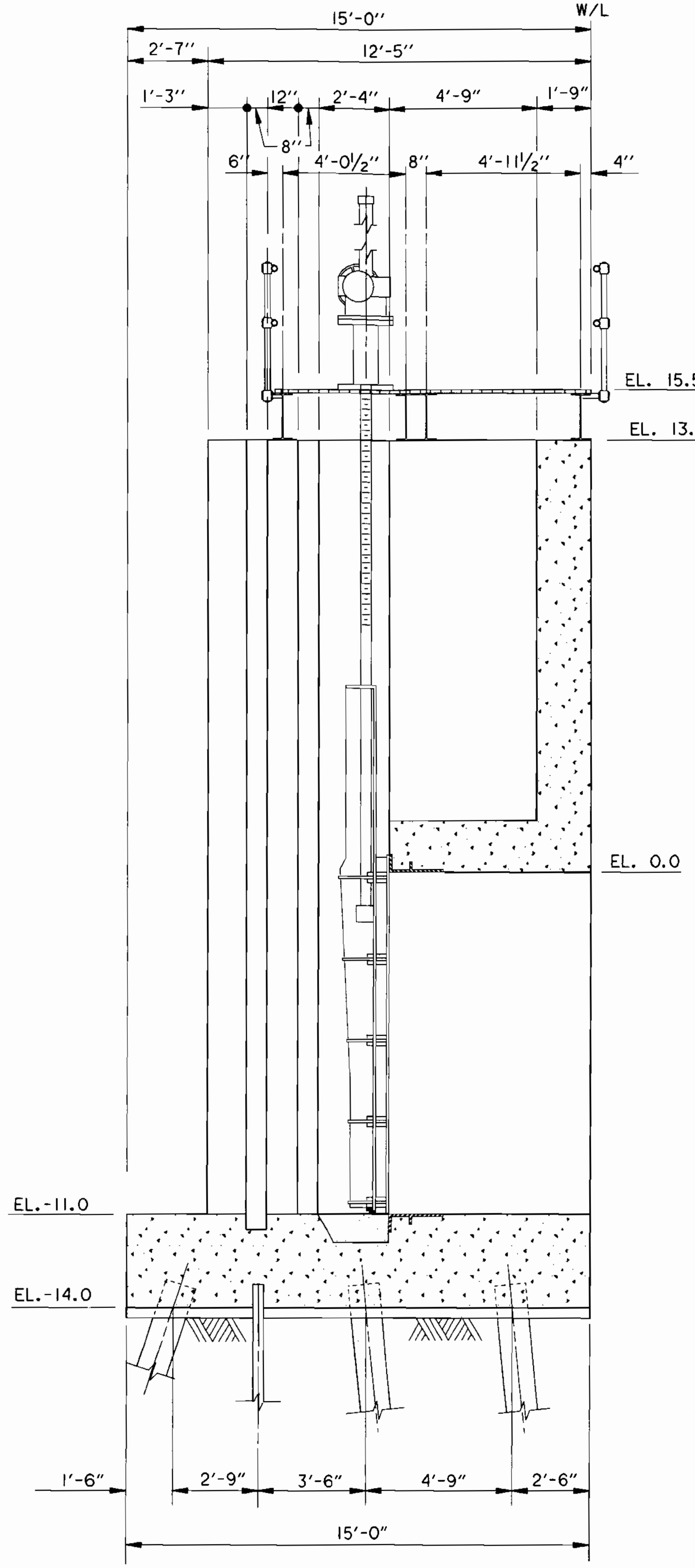
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W.	PLOT SCALE: 48	PLOT DATE: 9 JAN 95	CADD FILE: 40511N10.DGN
DRAWN BY: P.J.S./V.A.C.			FILE NO. H-2-40511
CHECKED BY: W.O.B.	DATE: JAN 1995		

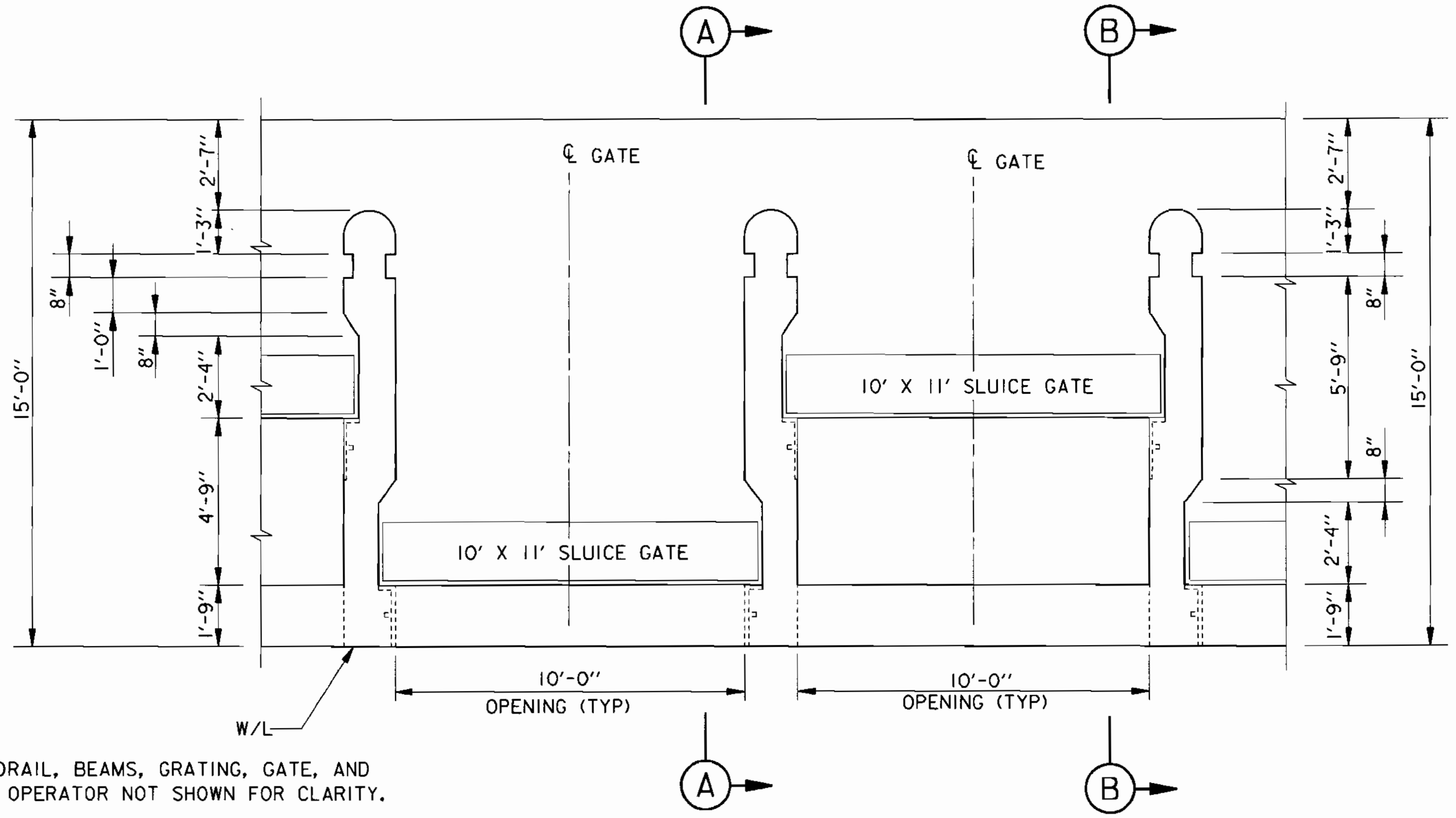
C•A•D



SECTION (A)

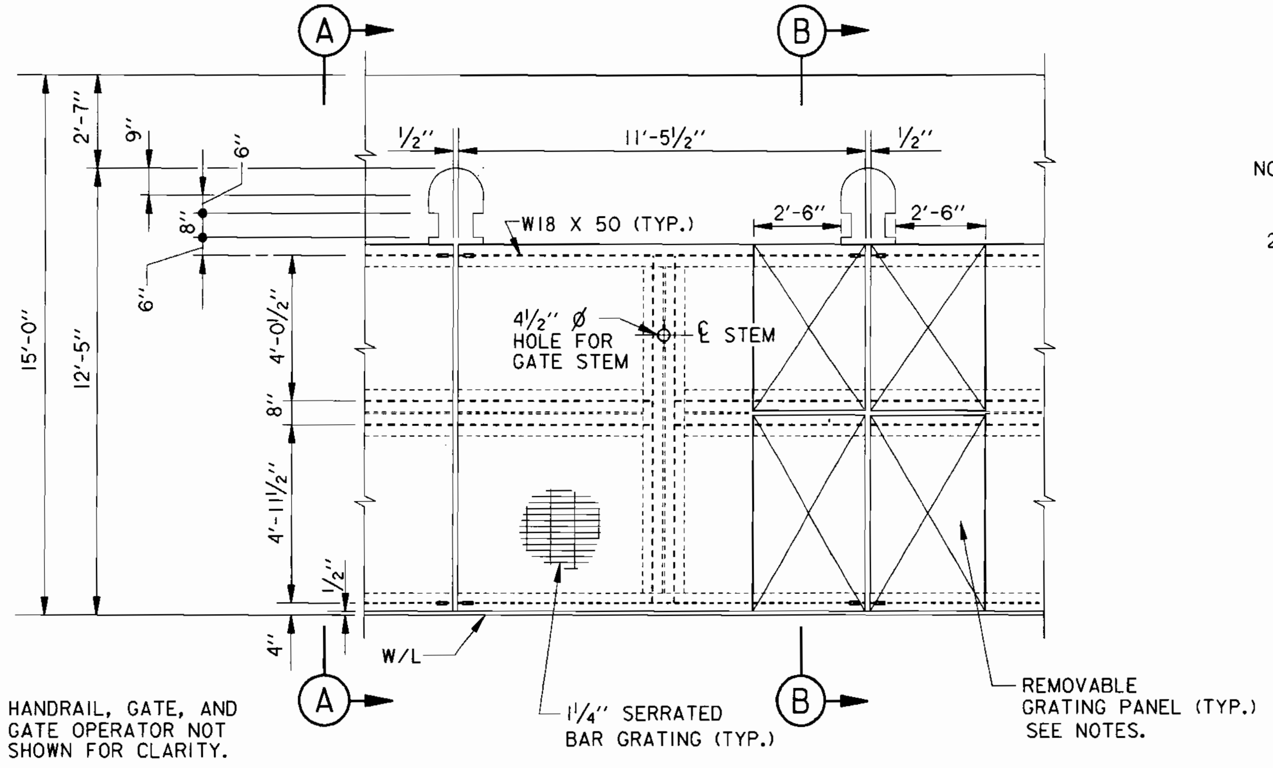


SECTION (B)



HANDRAIL, BEAMS, GRATING, GATE, AND GATE OPERATOR NOT SHOWN FOR CLARITY.

PARTIAL PLAN AT TOP OF OPERATING FLOOR - EL.13.90



HANDRAIL, GATE, AND GATE OPERATOR NOT SHOWN FOR CLARITY.

PARTIAL PLAN AT TOP OF GRATING - EL.15.50

SCALE: 3/8" = 1'- 0"

12" 0 2' 4' 6' 8' 10'

NOTES:
1. SECTIONS OF GRATING PANEL REMOVABLE TO ALLOW ACCESS FOR INSPECTION PURPOSES.
2. OPERATING FLOOR PANELS REMOVABLE VIA BOLTED CONNECTIONS TO CONCRETE MONOLITHS.

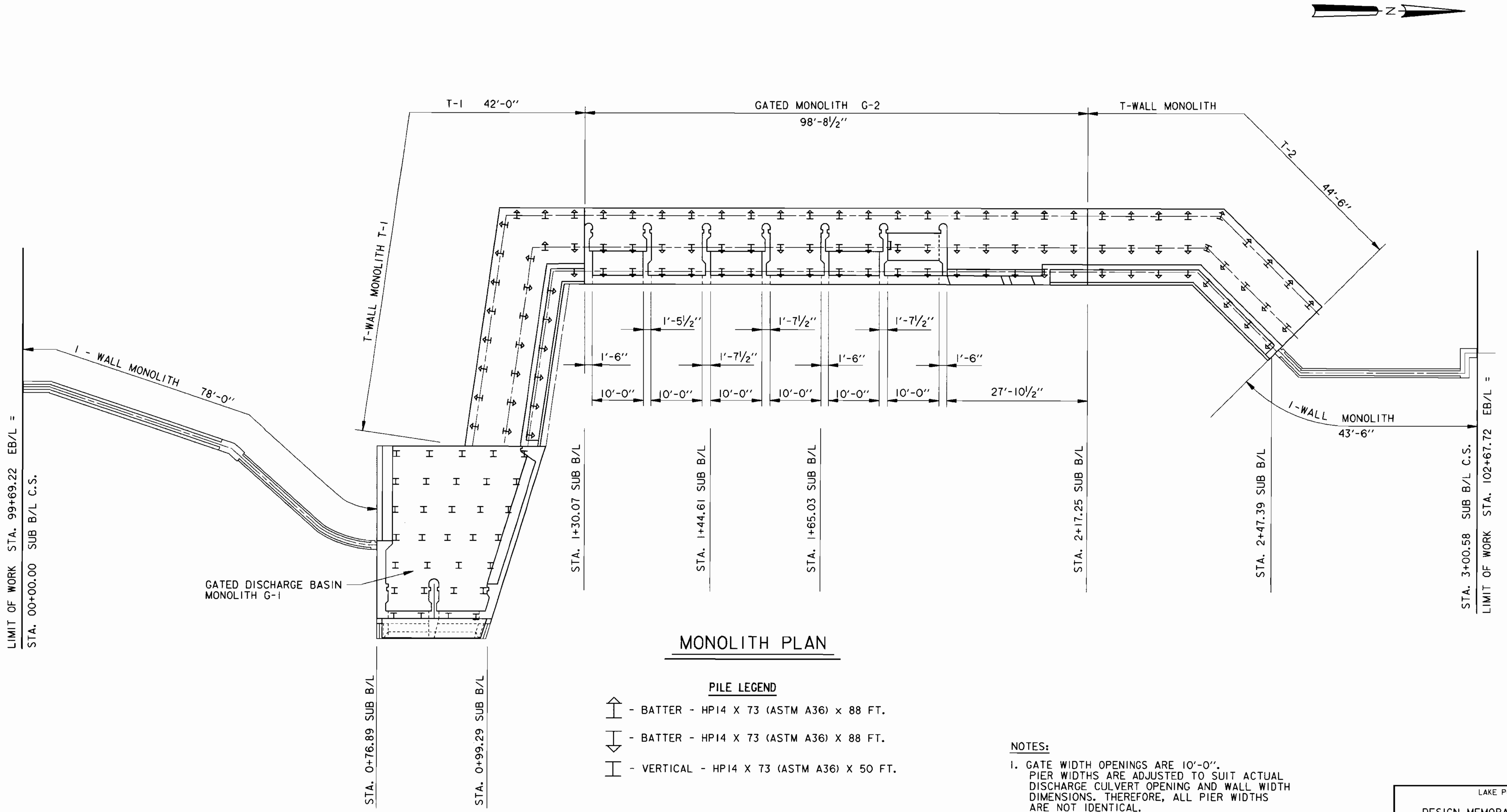
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

OPERATING FLOOR

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W.	PLOT SCALE: 32	PLOT DATE: 9 JAN 95	CADD FILE: 40511P06.DGN
DRAWN BY: L.V.M./V.A.C.			FILE NO. H-2-40511
CHECKED BY: W.O.B.	DATE: JAN 1995		

PLATE 9



MONOLITH PLAN

PILE LEGEND

- BATTER - HP14 X 73 (ASTM A36) x 88 FT.
- BATTER - HP14 X 73 (ASTM A36) X 88 FT.
- VERTICAL - HP14 X 73 (ASTM A36) X 50 FT.

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

NOTES:

1. GATE WIDTH OPENINGS ARE 10'-0". PIER WIDTHS ARE ADJUSTED TO SUIT ACTUAL DISCHARGE CULVERT OPENING AND WALL WIDTH DIMENSIONS. THEREFORE, ALL PIER WIDTHS ARE NOT IDENTICAL.
2. EQUIPMENT DECK LEVEL NOT SHOWN FOR CLARITY, SEE DWG. 4.
3. SEE FOUNDATION DESIGN PLATES FOR PILE TYPE, LOCATION, AND BATTER.
4. LENGTHS ARE ALONG WALL LINE.

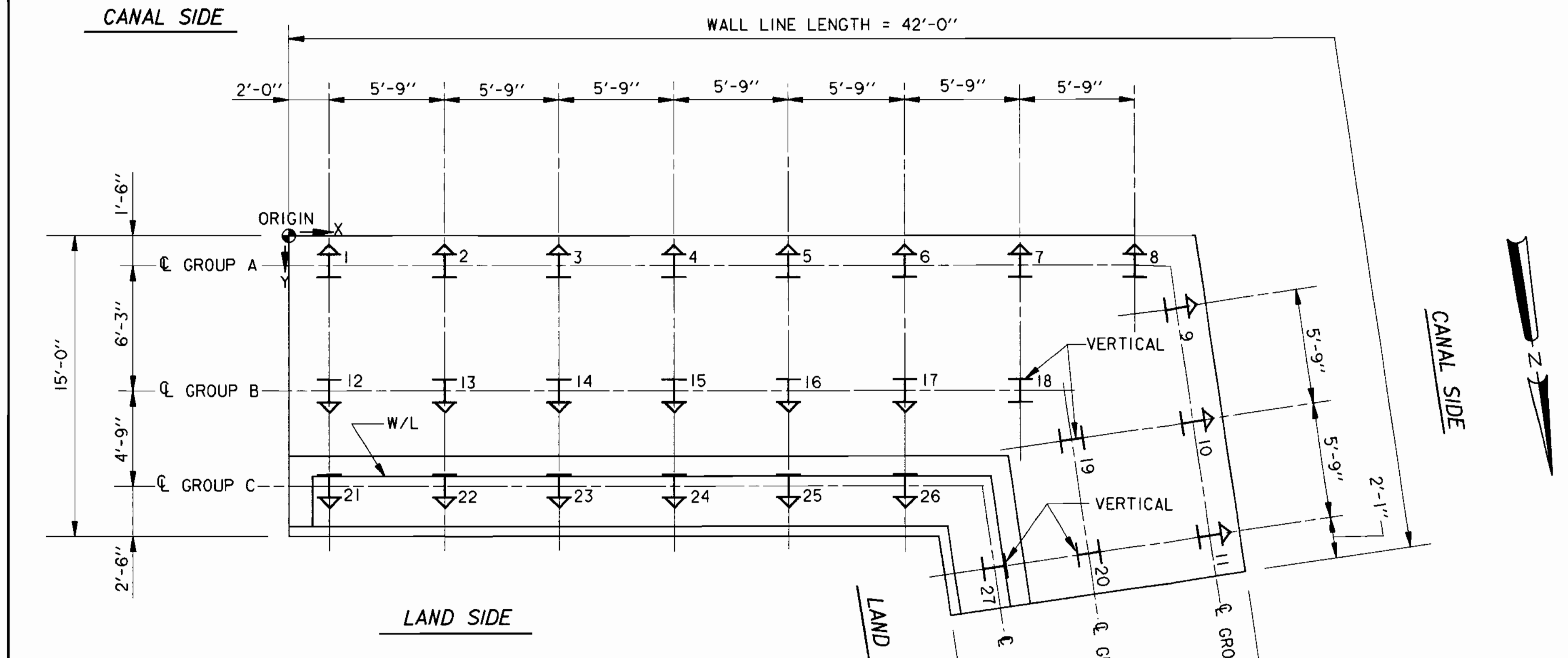
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

MONOLITH AND FOUNDATION PLAN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 120	PLOT DATE: 9 JAN 95	CADD FILE: 40511N22.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 10

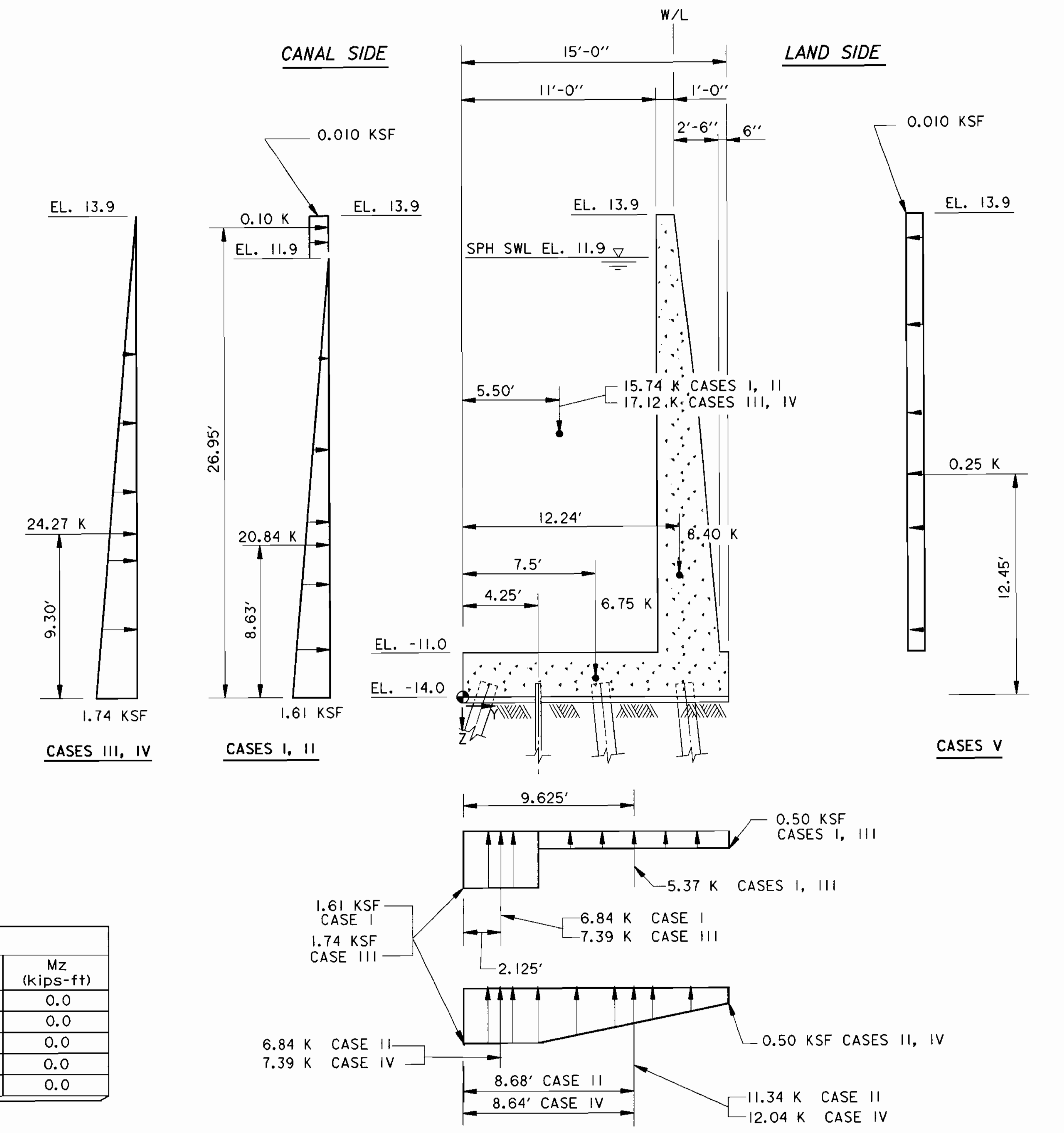


PILE LEGEND

- 4V on 1H BATTER - GROUP A - HP14 X 73 (ASTM A36) x 88 FT.
- 8V on 1H BATTER (U.N.O.) - GROUP B - HP14 X 73 (ASTM A36) X 88 FT.
- 6V on 1H BATTER (U.N.O.) - GROUP C - HP14 X 73 (ASTM A36) X 88 FT.

PILE LAYOUT - MONOLITH T-1

SCALE: 1/4" = 1'- 0"



NOTE:
LOADS ARE PER FT. OF WALL LENGTH

TOTAL APPLIED LOADS						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	-136.6	750.7	860.9	14844.1	-18247.5	15531.7
II	-136.6	750.7	611.3	12644.0	-13067.7	15531.7
III •	-121.1	665.4	670.7	12500.8	-14043.4	13766.8
IV •	-121.1	665.4	464.7	10649.0	-9770.1	13766.8
V	1.6	-9.0	677.3	7002.8	-14913.7	-185.0

• = APPLIED LOADS REDUCED TO 75% OF ACTUAL LOADS

LOAD PER FT. (PERPENDICULAR TO WALL)						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	0.0	20.9	18.7	355.0	0.0	0.0
II	0.0	20.9	12.7	309.0	0.0	0.0
III	0.0	24.7	19.5	404.0	0.0	0.0
IV	0.0	24.7	12.9	352.0	0.0	0.0
V	0.0	0.25	15.2	151.0	0.0	0.0

LOAD CASES

CASE I - CANAL SWL EL. 11.9 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE II - CANAL SWL EL. 11.9 NGVD, STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE III - CANAL SWL EL. 13.9 NGVD NO WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE IV - CANAL SWL EL. 13.9 NGVD NO WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE V - NO HYDROSTATIC LOAD, OPERATING WIND LOAD FROM PROTECTED SIDE.

SUMMARY OF PILE ANALYSIS (SEE NOTE 1)									
LOAD CASE	MAX. COMP.	% OF ALLOWABLE ALF (2)	% OF ALLOWABLE CBF (3)	MAX. TENSION	% OF ALLOWABLE ALF (2)	% OF ALLOWABLE CBF (3)	MAX. PILE CAP MOVEMENT X (in)	MAX. PILE CAP MOVEMENT Y (in)	MAX. PILE CAP MOVEMENT Z (in)
I	21	0.89	1.02	1	0.87	1.06	-.1687E+00	.4442E+00	.1015E-01
II	21	0.78	0.98	1	0.98	1.09	-.1675E+00	.4530E+00	.6097E-03
III	21	0.77	0.89	1	0.81	0.95	-.1485E+00	.3940E+00	.4591E-02
IV	21	0.68	0.86	1	0.90	0.98	-.1476E+00	.4019E+00	.2925E-02
V	27	0.36	0.18	8	0.01	0.02	-.1369E-02	.4846E-01	.1181E-01

NOTES:

1. PILE ANALYSIS RESULTS FROM COMPUTER PROGRAM CPGA IN THE CORPS LIBRARY (X0080).
2. ALLOWABLE LOAD FACTOR (ALF) % BASED ON MAX. SOIL PILE CAPACITY OF 132.0 kips IN COMPRESSION AND 122.0 kips IN TENSION FOR A FACTOR OF SAFETY OF 2.0. PILE LOAD TEST REQUIRED.
3. COMBINED BENDING FACTOR (CBF) % BASED ON INTERACTION EQUATIONS PRESENTED IN TECHNICAL MANUAL ITL-89-3 FOR CPGA COMPUTER PROGRAM.
4. FOR APPLIED LOADS THAT ARE NOT SYMMETRICAL ABOUT ORIGIN, MAX. PILE CAP MOVEMENT CORRESPONDS TO THE MONOLITH LOCATION WITH THE GREATEST MOVEMENT.

C•A•D

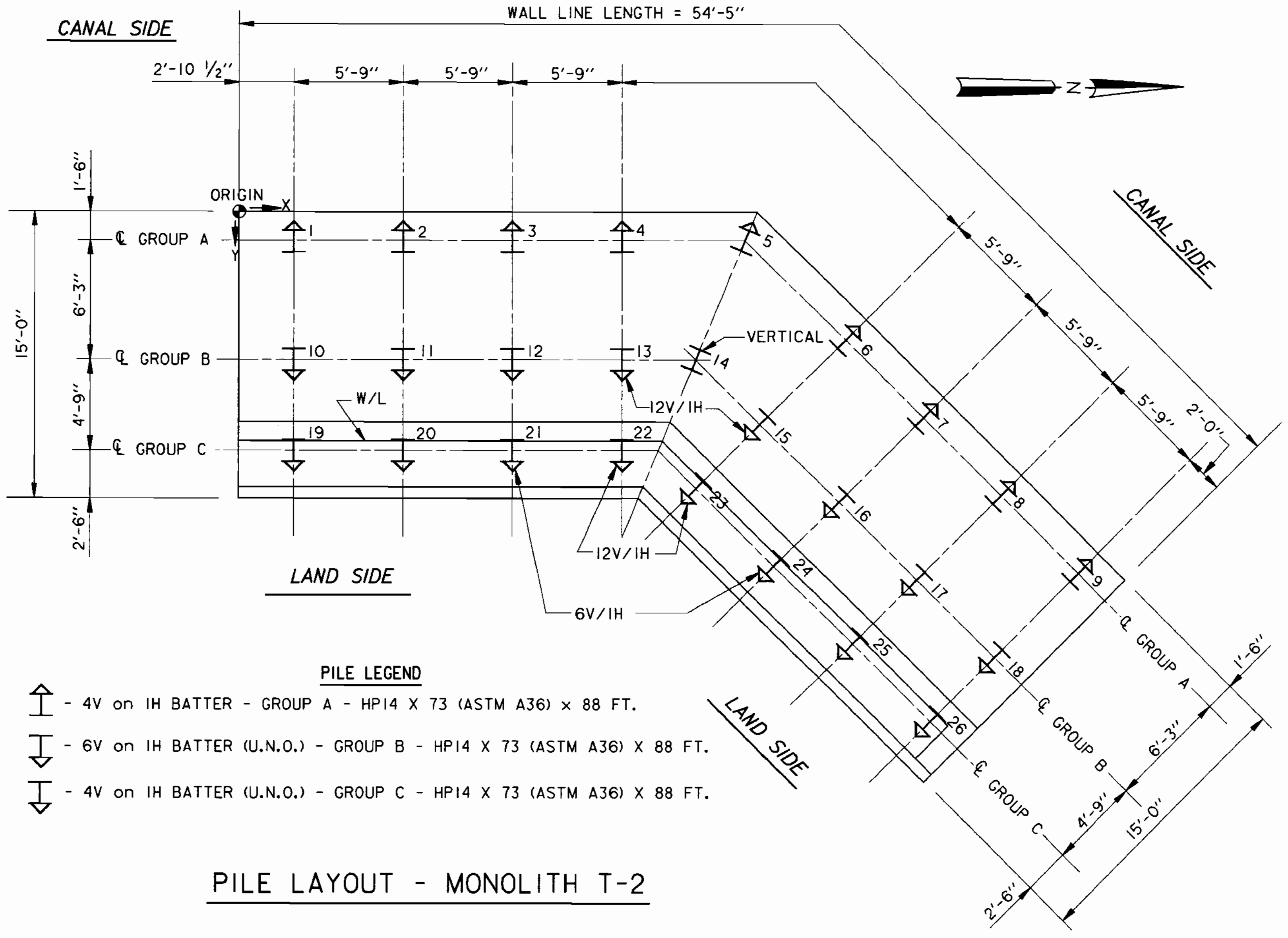
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

T-WALL MONOLITH T-1 FOUNDATION DESIGN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W. | DRAWN BY: J.C.M. | CHECKED BY: W.O.B.
PLOT SCALE: 48 | PLOT DATE: 9 JAN 95 | DATE: JAN 1995
CADD FILE: 4051IFSA.DGN | FILE NO. H-2-40511

PLATE 11

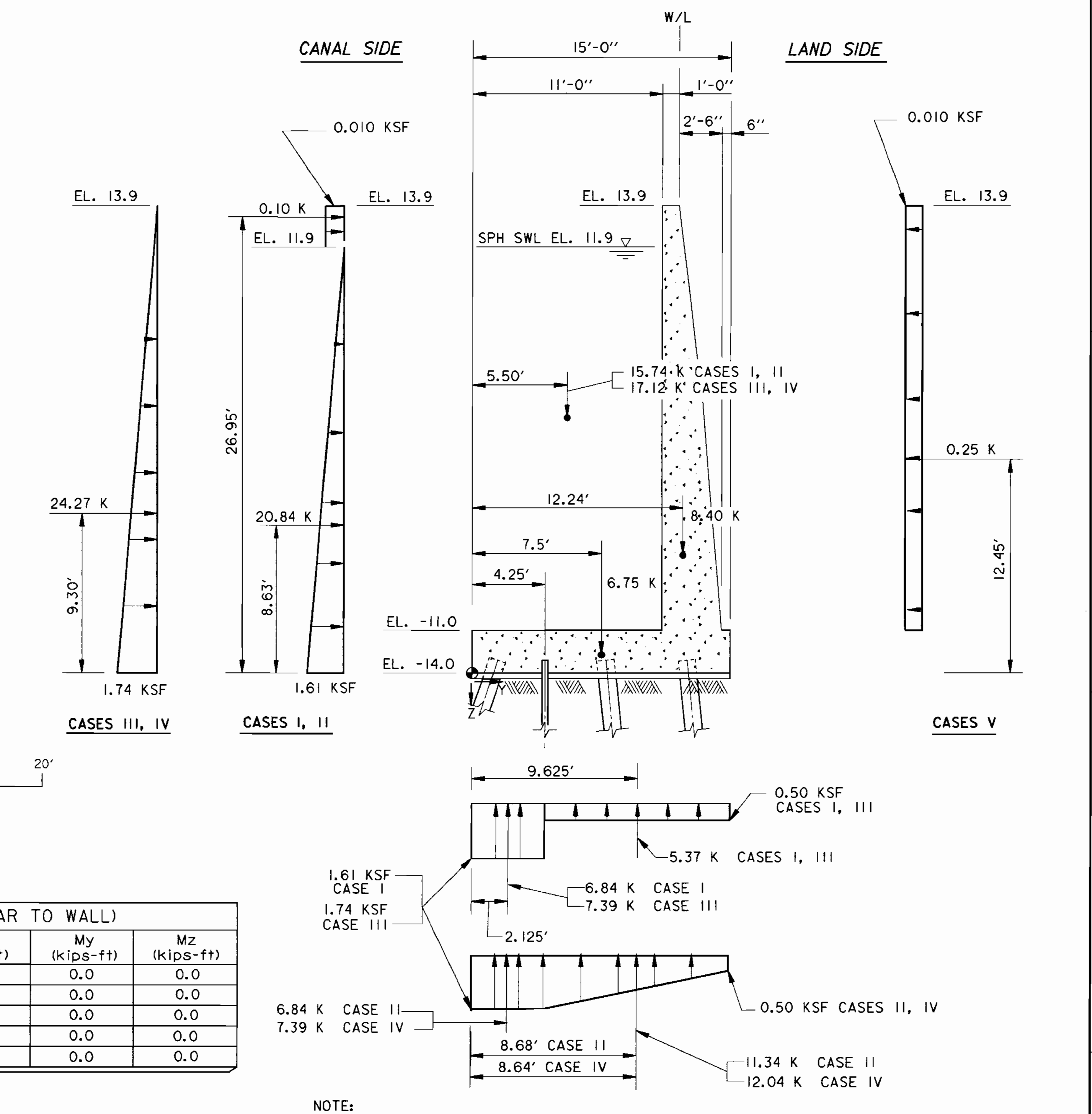


PILE LEGEND

- 4V on IH BATTER - GROUP A - HP14 X 73 (ASTM A36) x 88 FT.
- 6V on IH BATTER (U.N.O.) - GROUP B - HP14 X 73 (ASTM A36) X 88 FT.
- 4V on IH BATTER (U.N.O.) - GROUP C - HP14 X 73 (ASTM A36) X 88 FT.

PILE LAYOUT - MONOLITH T-2

SCALE: 1/4" = 1'- 0"



NOTE:
LOADS ARE PER FT. OF WALL LENGTH

TOTAL APPLIED LOADS						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	-359.1	867.0	958.8	20449.9	-19631.4	25337.0
II	-359.1	867.0	667.2	16605.6	-12778.8	25337.0
III •	-318.3	768.5	748.3	16924.2	-14987.6	22457.8
IV •	-318.3	768.5	507.7	13727.2	-9334.2	22457.8
V	4.3	-10.4	788.7	10776.9	-18759.4	-303.1

• = APPLIED LOADS REDUCED TO 75% OF ACTUAL LOADS

LOAD PER FT. (PERPENDICULAR TO WALL)						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	0.0	20.9	18.7	355.0	0.0	0.0
II	0.0	20.9	12.7	309.0	0.0	0.0
III	0.0	24.7	19.5	404.0	0.0	0.0
IV	0.0	24.7	12.9	352.0	0.0	0.0
V	0.0	0.25	15.2	151.0	0.0	0.0

LOAD CASES

CASE I - CANAL SWL EL. 11.9 NGVD,
STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE II - CANAL SWL EL. 11.9 NGVD,
STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE III - CANAL SWL EL. 13.9 NGVD
NO WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE IV - CANAL SWL EL. 13.9 NGVD
NO WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE V - NO HYDROSTATIC LOAD,
OPERATING WIND LOAD FROM PROTECTED SIDE.

SUMMARY OF PILE ANALYSIS (SEE NOTE 1)									
LOAD CASE	MAX. COMP.	% OF ALLOWABLE		MAX. TENSION	% OF ALLOWABLE		MAX. PILE CAP MOVEMENT		
		ALF (2)	CBF (3)		ALF (2)	CBF (3)	X (in)	Y (in)	Z (in)
I	26	1.07	1.09	1	0.63	0.91	-.1575E+00	.3802E+00	.3317E-01
II	25	0.95	1.07	1	0.74	0.96	-.1617E+00	.3906E+00	.1171E-01
III	26	0.92	0.92	4	0.62	0.83	-.1401E+00	.3383E+00	.2490E-01
IV	26	0.81	0.81	1	0.69	0.87	-.1438E+00	.3908E+00	.6700E-01
V	26	0.39	0.39	N/A	N/A	N/A	-.1628E-01	-.3908E-01	.6538E-01

NOTES:

1. PILE ANALYSIS RESULTS FROM COMPUTER PROGRAM CPGA IN THE CORPS LIBRARY (X0080). PILES FOR ANALYSIS WERE
2. ALLOWABLE LOAD FACTOR (ALF) % BASED ON MAX. SOIL PILE CAPACITY OF 132.0 kips IN COMPRESSION AND 122.0 kips IN TENSION FOR A FACTOR OF SAFETY OF 2.0. PILE LOAD TEST REQUIRED.
3. COMBINED BENDING FACTOR (CBF) % ALLOWABLE BASED ON INTERACTION EQUATIONS PRESENTED IN TECHNICAL MANUAL ITL-89-3 FOR CPGA COMPUTER PROGRAM.

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

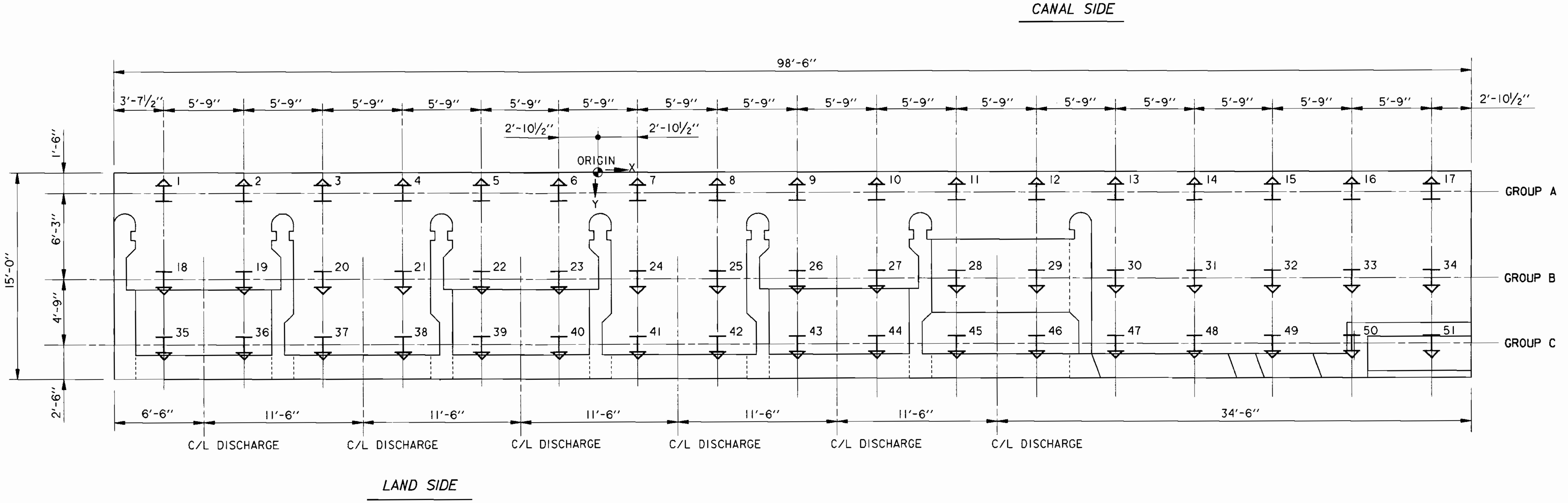
T-WALL MONOLITH T-2 FOUNDATION DESIGN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

C•A•D

DESIGNED BY: M.W.W.	PLOT SCALE: 48	PLOT DATE: 3 FEB 95	CADD FILE: 40511F6A.DGN	
DRAWN BY: J.C.M.			FILE NO. H-2-40511	
CHECKED BY: W.O.B.	DATE: JAN 1995			

PLATE 12



PILE LAYOUT - GATED MONOLITH G-2

PILE LEGEND

- 6V on IH BATTER - GROUP A - HP14 X 73 (ASTM A36) x 88 FT.
- 12V on IH BATTER - GROUP B - HP14 X 73 (ASTM A36) X 88 FT.
- 10V on IH BATTER - GROUP C - HP14 X 73 (ASTM A36) X 88 FT.

APPLIED LOADS						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	0.0	1517.0	2247.0	34769.0	-25935.0	29046.0
II	0.0	1517.0	1654.0	30129.0	-17643.0	29046.0
III •	0.0	1379.0	1775.0	30018.0	-20339.0	25158.0
IV •	0.0	1379.0	1281.0	26149.0	-13425.0	25158.0
V	0.0	1007.0	1634.0	21410.0	-21261.0	14099.0
VI	0.0	1007.0	1303.0	18818.0	-16629.0	14099.0
VII	0.0	0.0	1823.0	17253.0	-20897.0	0.0
VIII	0.0	1847.0	2247.0	37470.0	-25935.0	29046.0

• = APPLIED LOADS REDUCED TO 75% OF ACTUAL LOADS

SUMMARY OF PILE ANALYSIS (SEE NOTE 1)									
LOAD CASE	MAX. COMP.	% OF ALLOWABLE		MAX. TENSION	% OF ALLOWABLE		MAX. PILE CAP MOVEMENT		
		ALF (2)	CBF (3)		ALF (2)	CBF (3)	X (in)	Y (in)	Z (in)
I	51	0.99	0.87	17	0.87	0.81	-0.2055E-01	0.5161E+00	0.7930E-01
II	51	0.89	0.84	17	0.95	0.84	-0.2055E-01	0.5202E+00	0.6637E-01
III	51	0.86	0.75	17	0.81	0.71	-0.1516E-01	0.4444E+00	0.7111E-01
IV	51	0.78	0.73	17	0.87	0.74	-0.1515E-01	0.4479E+00	0.6030E-01
V	35	0.56	0.48	17	0.28	0.37	0.1018E-02	0.2733E+00	0.3551E-01
VI	35	0.50	0.46	17	0.32	0.38	0.1020E-02	0.2756E+00	0.2827E-01
VII	35	0.48	0.21	1	0.16	0.08	-0.9528E-04	-0.3811E-01	0.5956E-01
VIII	35	1.05	0.81	17	0.89	0.82	-0.7166E-02	0.5282E+00	0.7930E-01

NOTES:

1. FOUNDATION DESIGN BASED ON ORIGINAL CONSTRUCTION PLAN DIMENSIONS, NOT AS-BUILT DIMENSIONS.
2. PILE ANALYSIS RESULTS FROM COMPUTER PROGRAM CPGA IN THE CORPS LIBRARY (X0080).
3. ALLOWABLE LOAD FACTOR (ALF) % BASED ON MAX. SOIL PILE CAPACITY OF 132.0 kips IN COMPRESSION AND 122.0 kips IN TENSION FOR A FACTOR OF SAFETY OF 2.0. PILE LOAD TEST REQUIRED.
4. COMBINED BENDING FACTOR (CBF) % BASED ON INTERACTION EQUATIONS PRESENTED IN TECHNICAL MANUAL ITL-89-3 FOR CPGA COMPUTER PROGRAM.
5. FOR APPLIED LOADS THAT ARE NOT SYMMETRICAL ABOUT ORIGIN, MAX. PILE CAP MOVEMENT CORRESPONDS TO THE MONOLITH LOCATION WITH THE GREATEST MOVEMENT.

SCALE: 1/4" = 1'- 0"

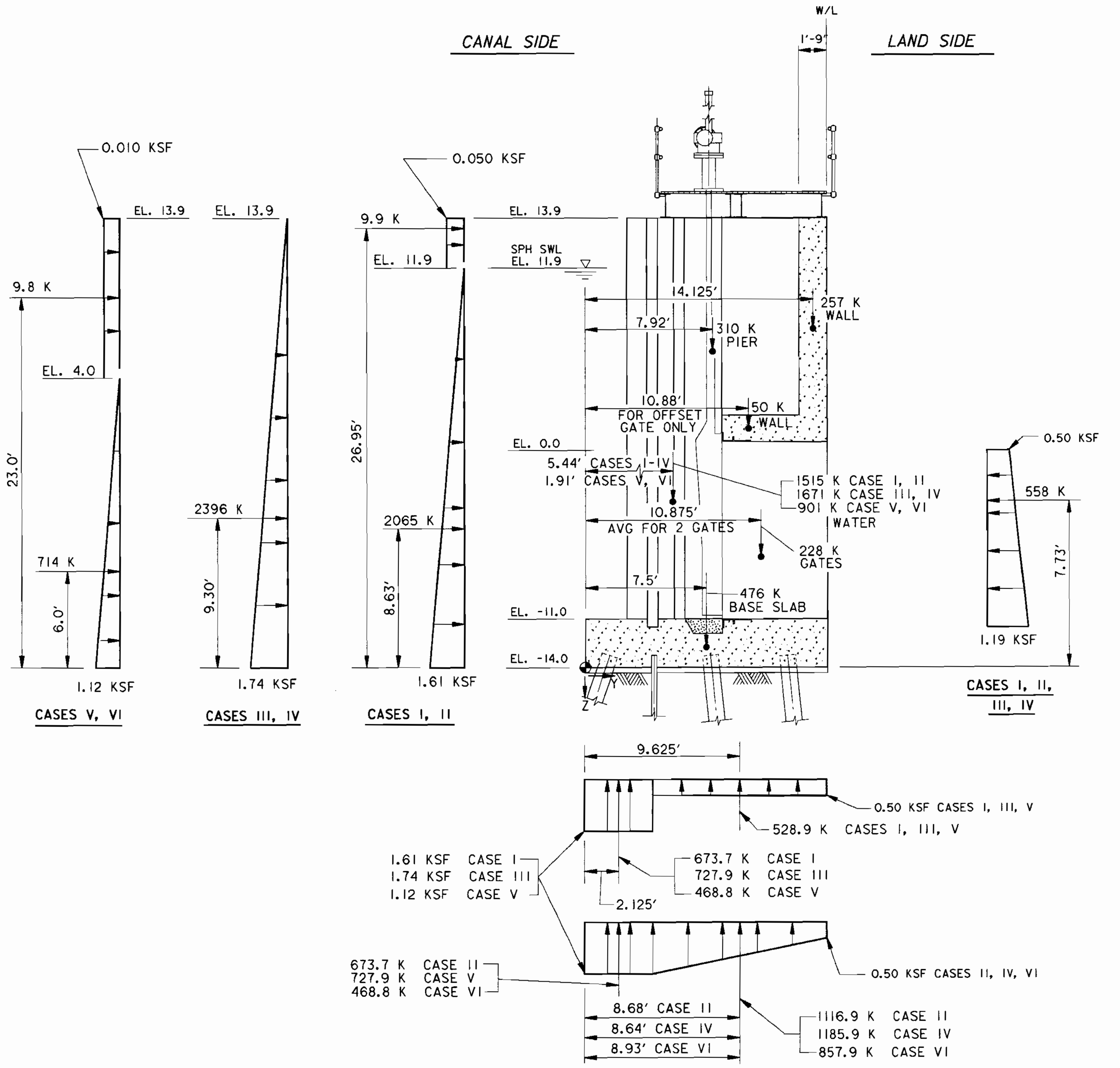
12" 0 5' 10' 15' 20'

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

GATED MONOLITH G-2 FOUNDATION DESIGN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: M.W.W. DRAWN BY: J.C.M. CHECKED BY: W.O.B.
PLOT SCALE: 48 | PLOT DATE: 3 FEB 95 | CADD FILE: 40511B01.DGN | DATE: JAN 1995 | FILE NO. H-2-40511



LOAD CASES

CASE I - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE II - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE III - GATE CLOSED, CANAL SWL EL. 13.9 NGVD SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE IV - GATE CLOSED, CANAL SWL EL. 13.9 NGVD SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE V - DEWATERING STOP LOGS INSTALLED, CANAL SWL EL. 4.0 NGVD, SWL INSIDE DISCHARGE CULVERTS EL.-11.00 NGVD, OPERATING WIND, IMPERVIOUS SHEET PILE CUT-OFF,

CASE VI - DEWATERING STOP LOGS INSTALLED, CANAL SWL EL. 4.0 NGVD, SWL INSIDE DISCHARGE CULVERTS EL.-11.00 NGVD, OPERATING WIND, PERVIOUS SHEET PILE CUT-OFF,

CASE VII - NO WATER LOADS. OPERATING WIND.

CASE VIII - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 1.0 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

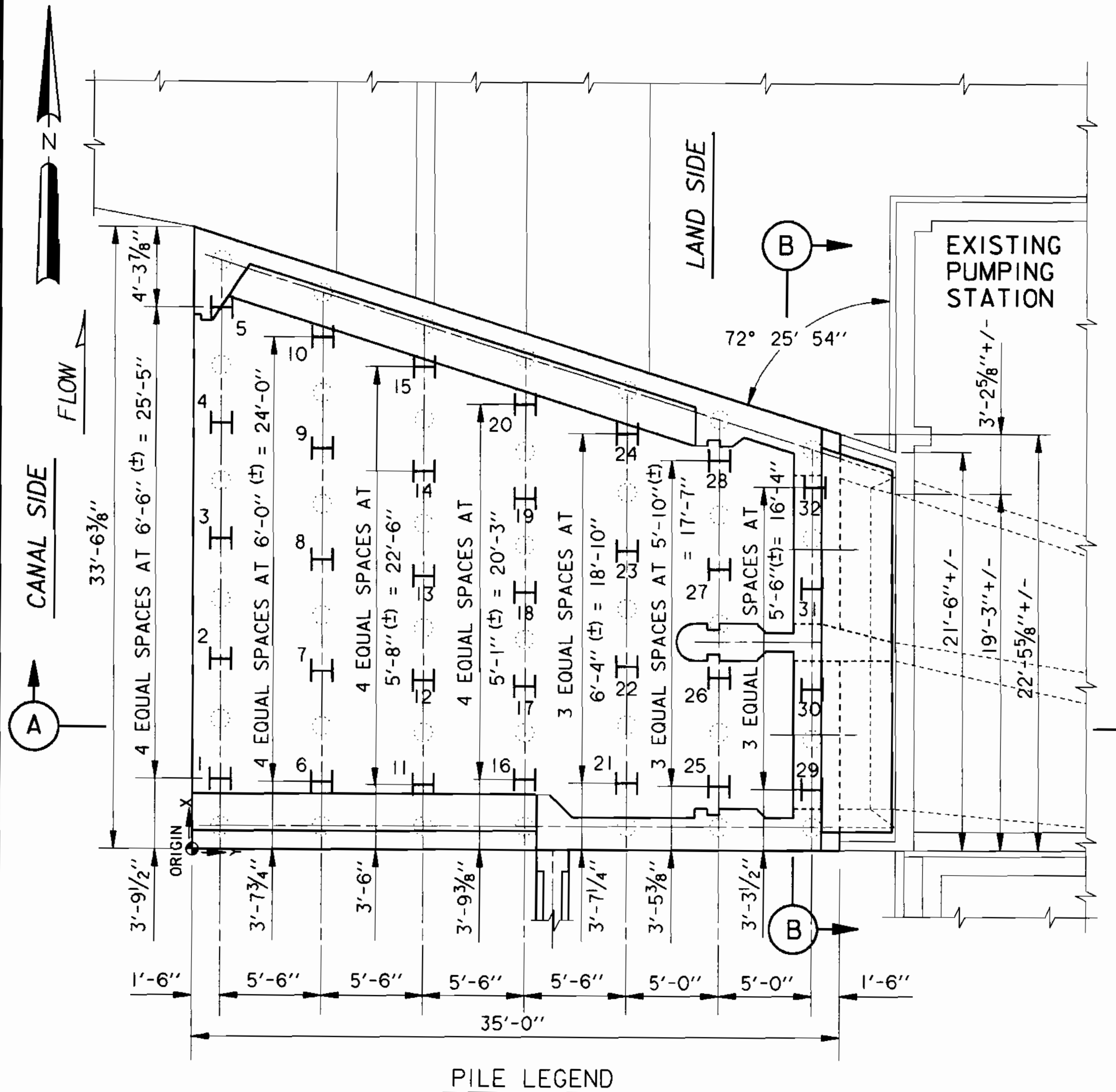
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

GATED MONOLITH G-2 FOUNDATION DESIGN LOADS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 48	PLOT DATE: 3 FEB 95	CADD FILE: 40511B02.DGN
DRAWN BY:	J.C.M.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 14



PILE LEGEND

ALL PILES VERTICAL - HP14 X 73 (ASTM A36) x 50 FT.

PILE LAYOUT - DISCHARGE BASIN

SCALE: 1" = 5'

LOAD CASES

CASE I - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 3.57 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE II - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 3.57 NGVD, STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

CASE III - GATE CLOSED, CANAL SWL EL. 13.9 NGVD SWL INSIDE DISCHARGE CULVERTS EL. 3.57 NGVD, STORM WIND LOAD, IMPERVIOUS SHEET PILE CUT-OFF.

CASE IV - GATE CLOSED, CANAL SWL EL. 13.9 NGVD SWL INSIDE DISCHARGE CULVERTS EL. 3.57 NGVD, STORM WIND LOAD, PERVIOUS SHEET PILE CUT-OFF.

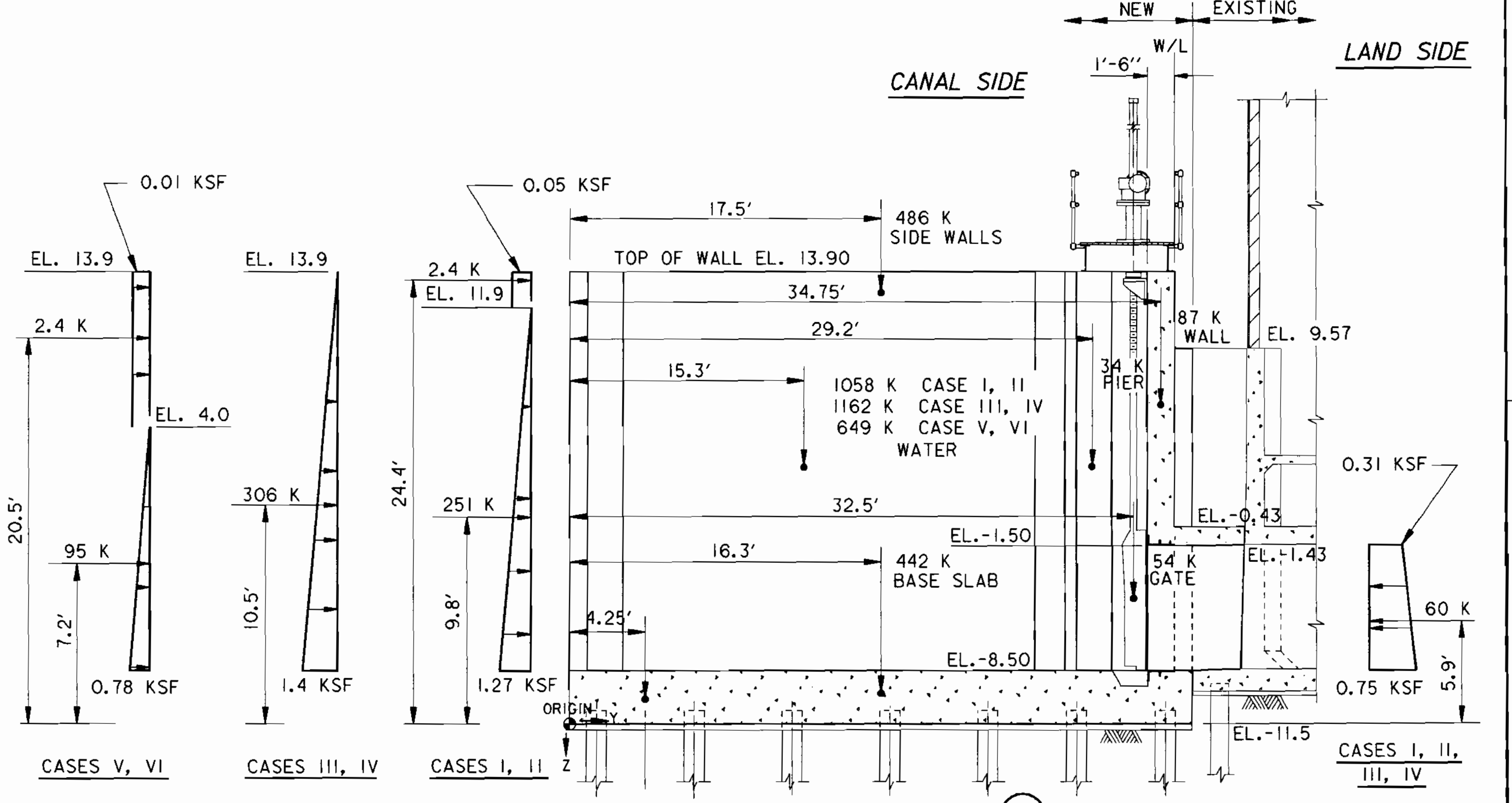
CASE V - DEWATERING STOP LOGS INSTALLED, CANAL SWL EL. 4.0 NGVD, SWL INSIDE DISCHARGE CULVERTS EL.-8.50 NGVD, OPERATING WIND, IMPERVIOUS SHEET PILE CUT-OFF.

CASE VI - DEWATERING STOP LOGS INSTALLED, CANAL SWL EL. 4.0 NGVD, SWL INSIDE DISCHARGE CULVERTS EL.-8.50 NGVD, OPERATING WIND, PERVIOUS SHEET PILE CUT-OFF.

CASE VII - NO WATER LOADS, OPERATING WIND, CONSTRUCTION CONDITIONS.

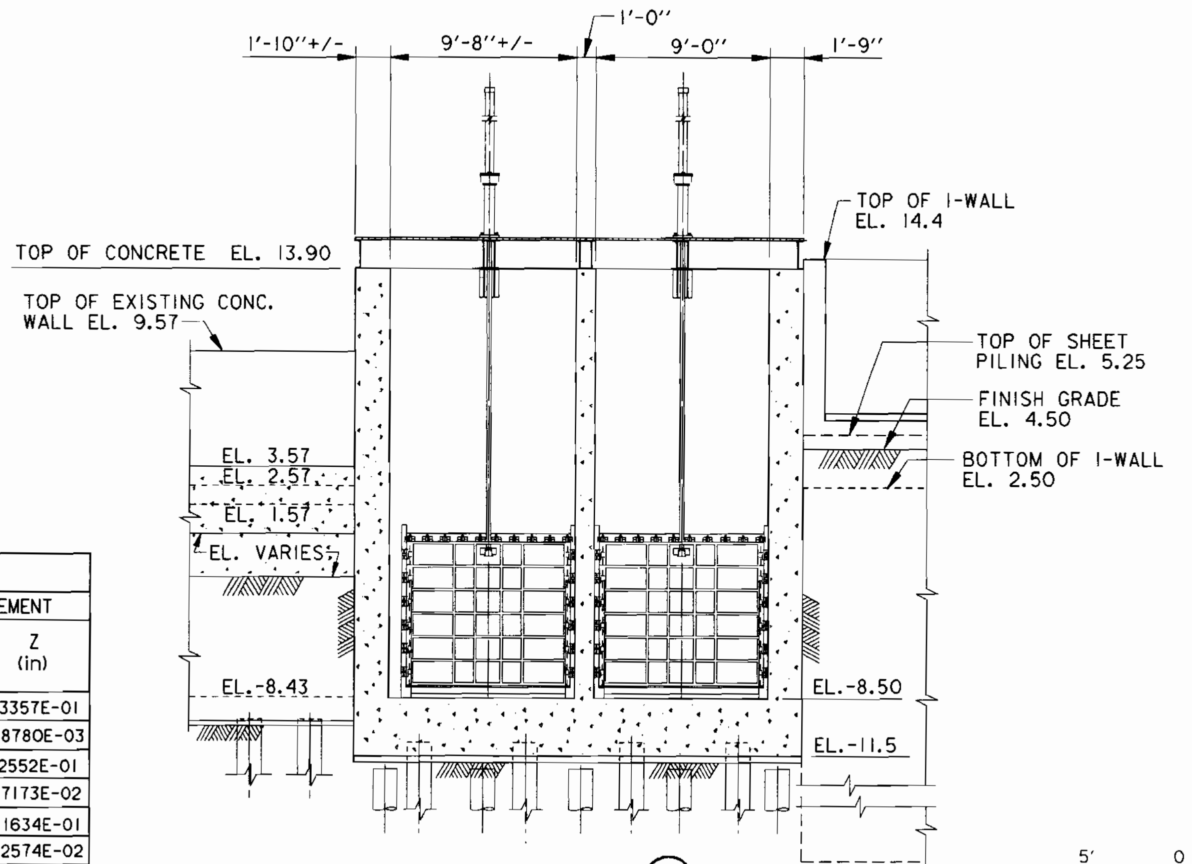
NOTES:

1. PILE ANALYSIS RESULTS FROM COMPUTER PROGRAM CPGA IN THE CORPS LIBRARY (X0080).
2. ALLOWABLE LOAD FACTOR (ALF) % BASED ON MAX. SOIL PILE CAPACITY OF 70. kips IN COMPRESSION AND 60. kips IN TENSION FOR A FACTOR OF SAFETY OF 2.0. PILE LOAD TEST REQUIRED.
3. COMBINED BENDING FACTOR (CBF) % BASED ON INTERACTION EQUATIONS PRESENTED IN TECHNICAL MANUAL ITL-89-3 FOR CPGA COMPUTER PROGRAM.
4. FOR APPLIED LOADS THAT ARE NOT SYMMETRICAL ABOUT ORIGIN, MAX. PILE CAP MOVEMENT CORRESPONDS TO THE MONOLITH LOCATION WITH THE GREATEST MOVEMENT.



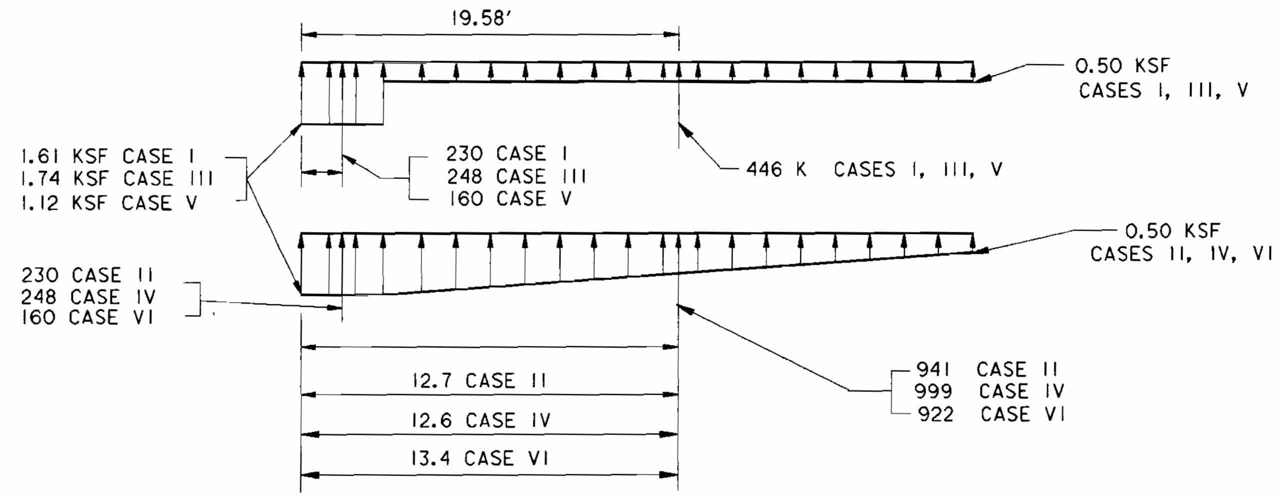
SECTION (A)

SCALE: 1" = 5'



SECTION (B)

SCALE: 1" = 5'



SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

APPLIED LOADS						
LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
I	.0	327.0	1485.0	28483.0	-19892.0	5808.0
II	.0	327.0	990.0	25300.0	-13989.0	5808.0
III •	.0	308.0	1179.0	22494.0	-16680.0	4673.0
IV •	.0	308.0	763.0	19609.0	-12601.0	4673.0
V	.0	147.0	1146.0	22330.0	-16205.0	2470.0
VI	.0	147.0	870.0	21397.0	-12285.0	2470.0
VII	.0	.0	1103.0	21481.0	-15594.0	.0

• = APPLIED LOADS REDUCED TO 75% OF ACTUAL LOADS

SUMMARY OF PILE ANALYSIS (SEE NOTE 1)

LOAD CASE	MAX. COMP.	% OF ALLOWABLE ALF (2)	% OF ALLOWABLE CBF (3)	MAX. TENSION	% OF ALLOWABLE ALF (2)	% OF ALLOWABLE CBF (3)	MAX. PILE CAP MOVEMENT X (in)	MAX. PILE CAP MOVEMENT Y (in)	MAX. PILE CAP MOVEMENT Z (in)
I	15	.81	.47	N/A	N/A	N/A	.8868E-01	.1380E+00	.3357E-01
II	32	1.03	.57	1	.26	.32	.8932E-01	.1212E+00	-.8780E-03
III	15	.72	.36	N/A	N/A	N/A	.2668E-01	.1757E+00	.2552E-01
IV	32	.89	.41	1	.47	.29	.2940E-01	.1602E+00	-.7173E-02
V	32	.71	.29	N/A	N/A	N/A	.3245E-01	.7297E-01	.1634E-01
VI	32	.87	.31	N/A	N/A	N/A	.3123E-01	.6357E-01	-.2574E-02
VII	32	.71	.15	N/A	N/A	N/A	.3993E-02	8309E-02	.1030E-01

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

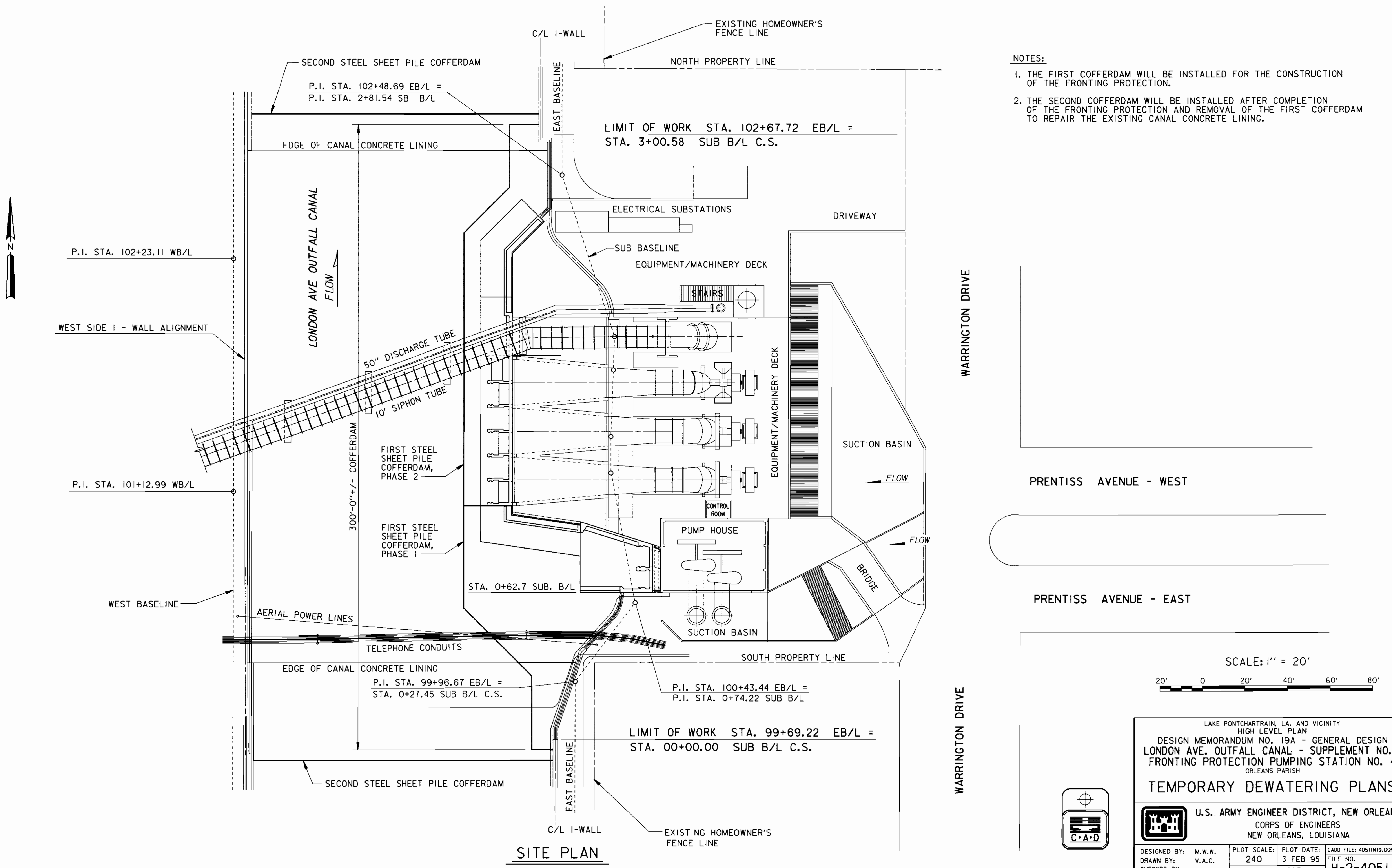
GATED DISCHARGE BASIN MONOLITH G-1 FOUNDATION DESIGN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

C•A•D

DESIGNED BY:	M.W.W.	PLOT SCALE: 60	PLOT DATE: 3 FEB 95	CADD FILE: 40511P02.DGN
DRAWN BY:	LVM/JCM			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 15



SITE PLAN

NOTES:

1. THE FIRST COFFERDAM WILL BE INSTALLED FOR THE CONSTRUCTION OF THE FRONTING PROTECTION.
2. THE SECOND COFFERDAM WILL BE INSTALLED AFTER COMPLETION OF THE FRONTING PROTECTION AND REMOVAL OF THE FIRST COFFERDAM TO REPAIR THE EXISTING CANAL CONCRETE LINING.

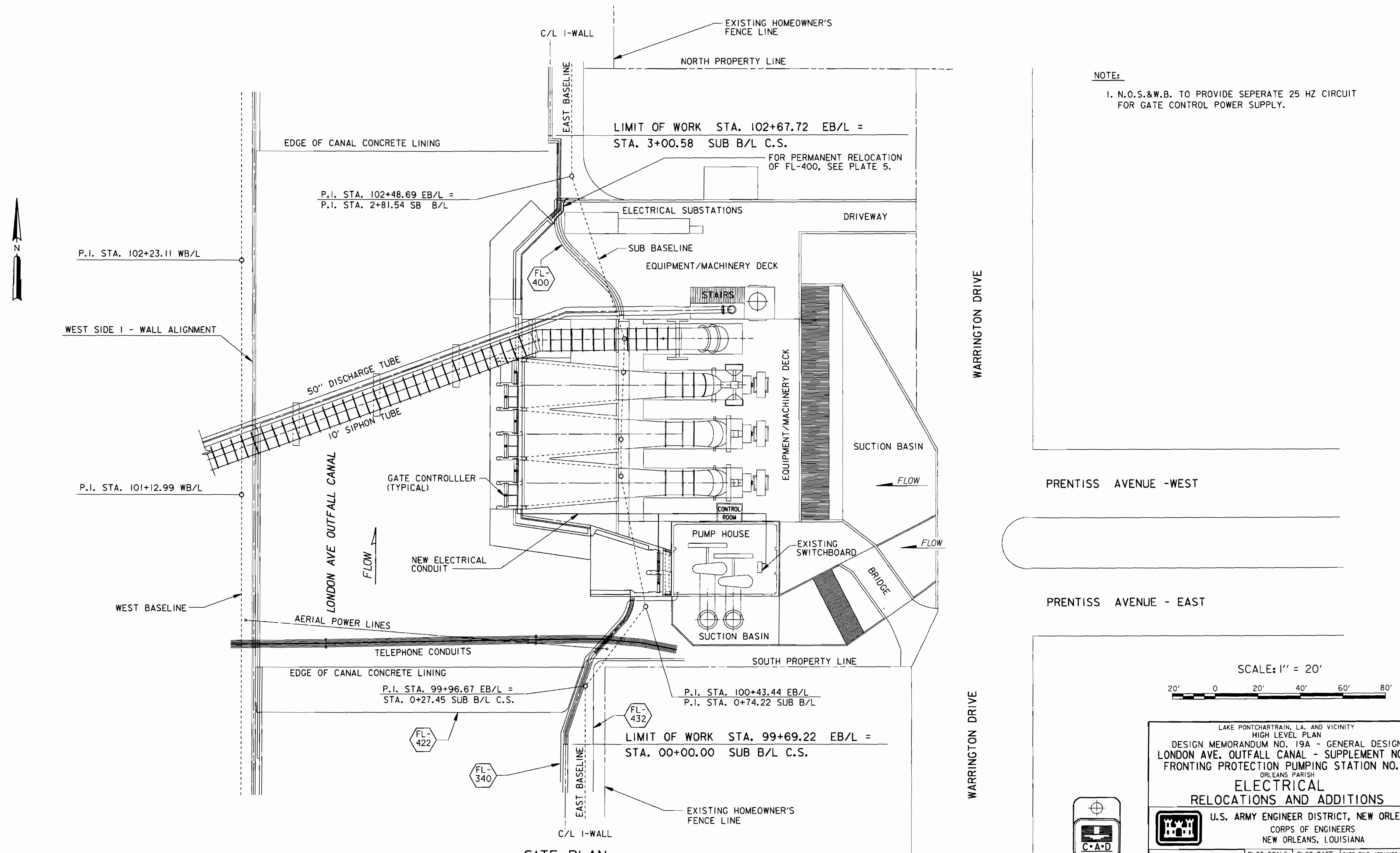
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

TEMPORARY DEWATERING PLANS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 240	PLOT DATE: 3 FEB 95	CADD FILE: 40511N19.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

PLATE 16



SITE PLAN

NOTE:

1. N.O.S.&W.B. TO PROVIDE SEPERATE 25 HZ CIRCUIT FOR GATE CONTROL POWER SUPPLY.

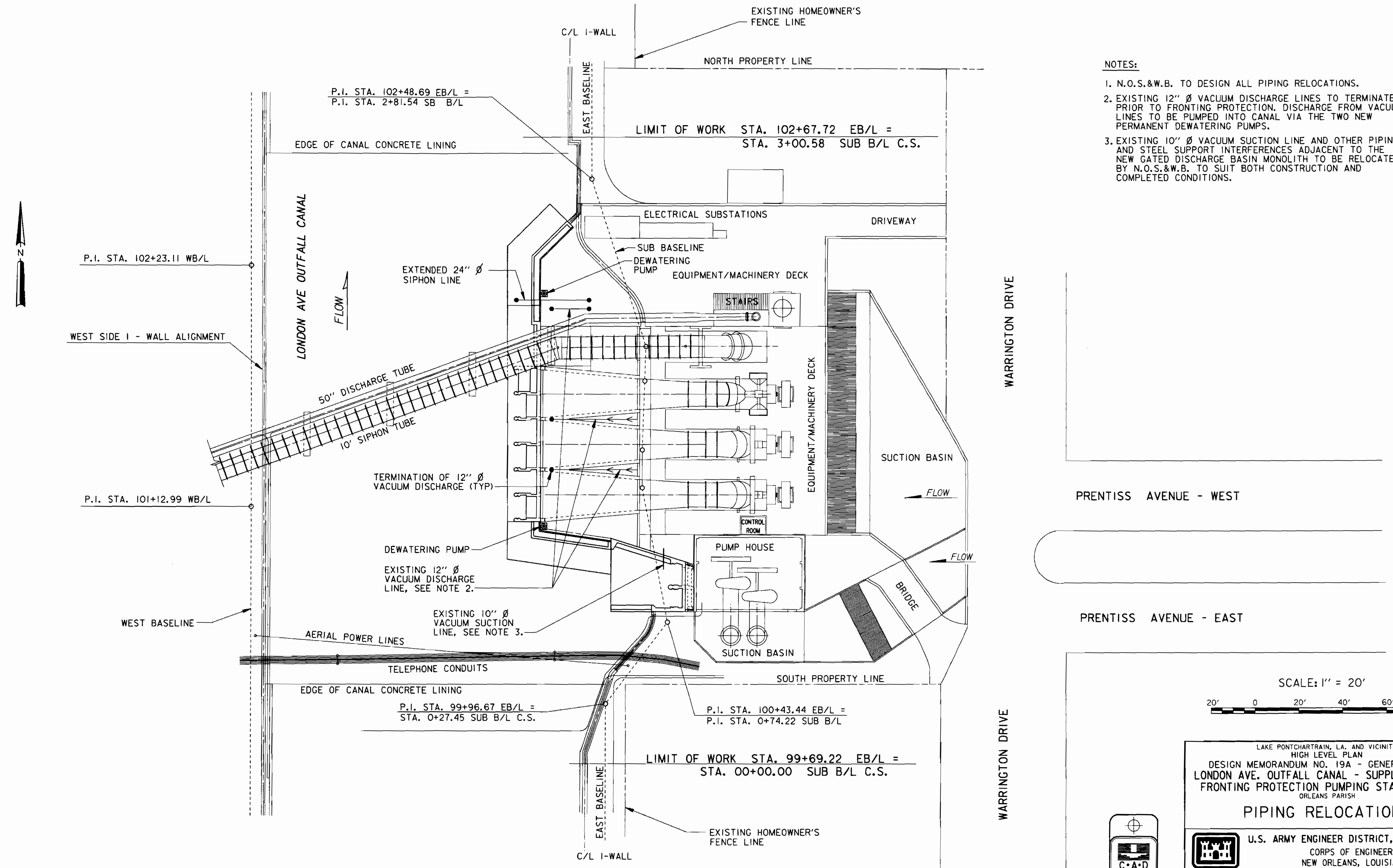
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

ELECTRICAL
RELOCATIONS AND ADDITIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 240	PLOT DATE: 3 FEB 95	CADD FILE: 40511IN20.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		

C•A•D



SITE PLAN

NOTES:

1. N.O.S.&W.B. TO DESIGN ALL PIPING RELOCATIONS.
2. EXISTING 12" Ø VACUUM DISCHARGE LINES TO TERMINATE PRIOR TO FRONTING PROTECTION. DISCHARGE FROM VACUUM LINES TO BE PUMPED INTO CANAL VIA THE TWO NEW PERMANENT DEWATERING PUMPS.
3. EXISTING 10" Ø VACUUM SUCTION LINE AND OTHER PIPING AND STEEL SUPPORT INTERFERENCES ADJACENT TO THE NEW GATED DISCHARGE BASIN MONOLITH TO BE RELOCATED BY N.O.S.&W.B. TO SUIT BOTH CONSTRUCTION AND COMPLETED CONDITIONS.

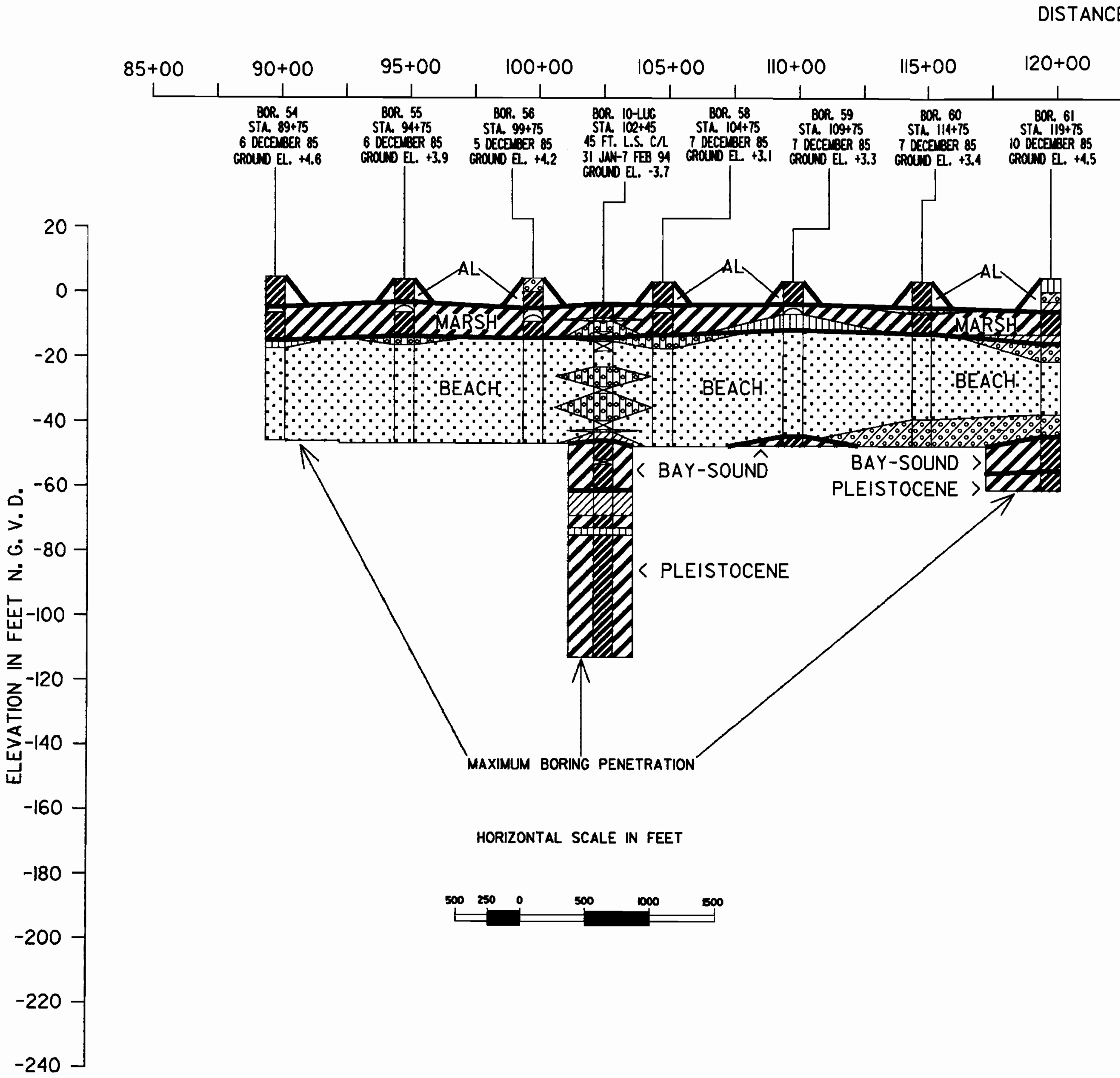
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

PIPING RELOCATIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

C·A·D

DESIGNED BY:	M.W.W.	PLOT SCALE: 240	PLOT DATE: 3 FEB 95	CADD FILE: 40511N24.DGN
DRAWN BY:	V.A.C.			FILE NO. H-2-40511
CHECKED BY:	W.O.B.	DATE: JAN 1995		



SOIL LEGEND

- CH - FAT CLAY
- CL - LEAN CLAY
- PT - PEAT
- ML - SILT
- SC - CLAYEY SAND
- SP - SAND POORLY-GRADED
- Wd - WOOD
- NO SAMPLE

BAY-SOUND - PREDOMINANTLY SILTS AND SILTY SANDS WITH SOME CLAY LAYERS

BEACH - PREDOMINANTLY SANDS AND SILTY SANDS WITH SHELL FRAGMENTS

PLEISTOCENE - STIFF TO VERY STIFF OXIDIZED CLAYS INTERBEDDED WITH LAYERS AND LENSES OF SILTS AND SANDS

MARSH - PREDOMINANTLY ORGANIC CLAYS, FAT CLAYS, AND PEATS WITH OCCASIONAL SAND AND SILT LAYERS

AL = ARTIFICIAL LEVEE

The parameters for defining SP and SM have changed over the past few years. (As per LMVD directive) Therefore, SM in recent borings may be equivalent to SP in the older borings.

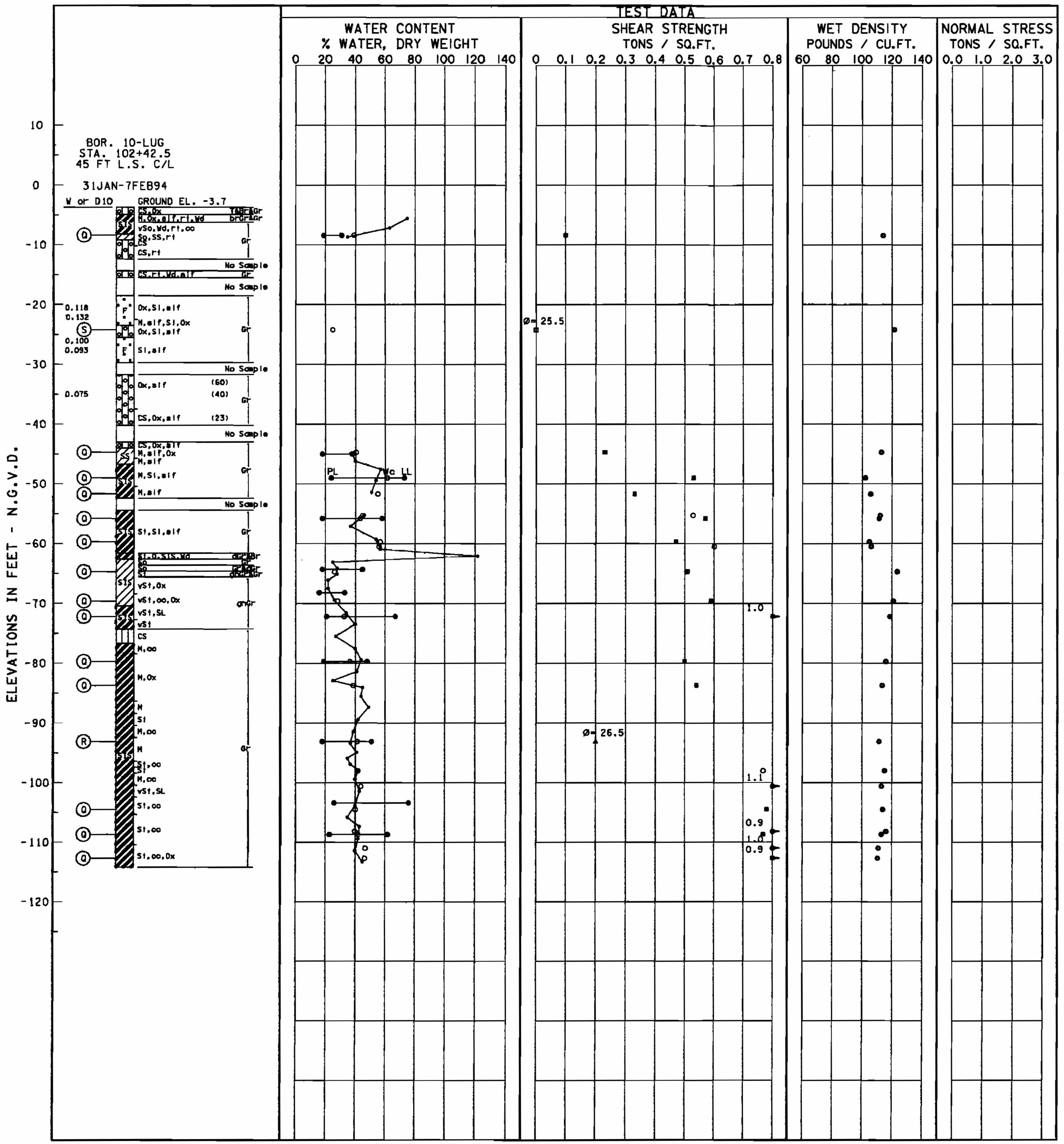
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
LONDON AVE OUTFALL CANAL

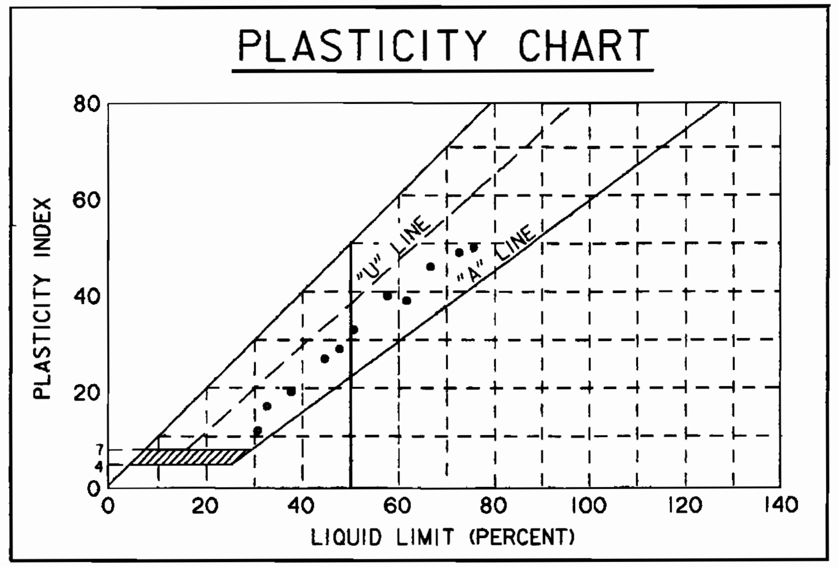
SOIL AND GEOLOGIC PROFILE
PUMPING STA NO. 4

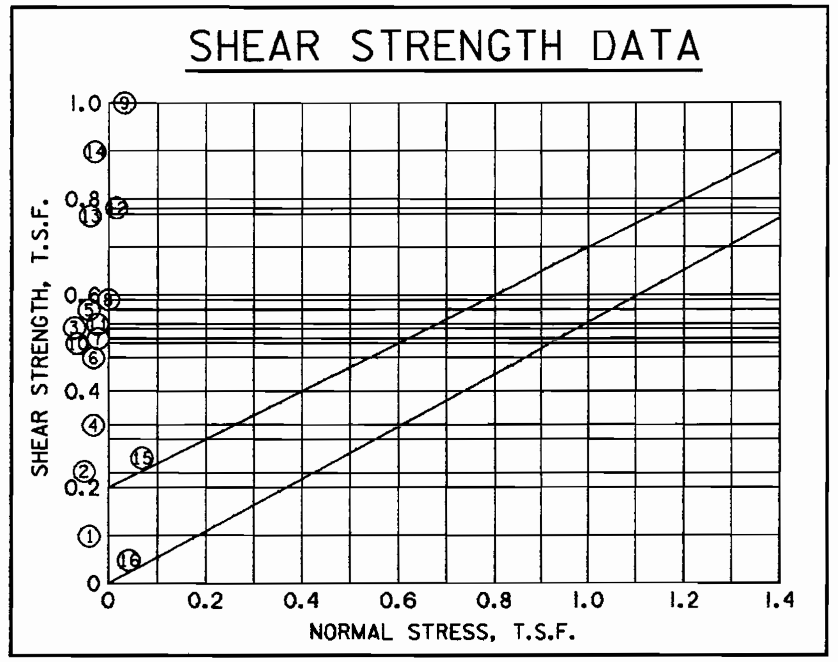
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: T. CREASY DRAWN BY: T. CREASY CHECKED BY: D. BRITSCH	PLOT SCALE: 40:1	PLOT DATE: DEC 94	CADD FILE: londonl.dgn
	DATE: DEC 94		FILE NO. H-2-40511

PLATE 19







TABULAR TEST DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH Φ	C - TSF	CLASS
1	-8.40	Q	0.0	0.130	CL
2	-44.70	Q	0.0	0.230	CL
3	-49.00	Q	0.0	0.530	CH
4	-51.70	Q	0.0	0.330	CH
5	-55.80	Q	0.0	0.570	CH
6	-59.70	Q	0.0	0.470	CH
7	-64.70	Q	0.0	0.510	CL
8	-69.60	Q	0.0	0.590	CL
9	-72.20	Q	0.0	1.000	CH
10	-79.70	Q	0.0	0.500	CL
11	-83.70	Q	0.0	0.540	CH
12	-104.50	Q	0.0	0.780	CH
13	-108.70	Q	0.0	0.761	CH
14	-112.70	Q	0.0	0.900	CH
15	-93.10	R	26.5	0.200	CH
16	-24.20	S	25.5	0.000	SM

NOTES

- O - (UC) UNCONFINED COMPRESSION TEST
- ■ - (Q) UNCONSOLIDATED - UNDRAINED TRIAXIAL SHEAR TEST
- ▲ - (R) CONSOLIDATED - UNDRAINED TRIAXIAL SHEAR TEST
- ⊡ - (S) CONSOLIDATED - DRAINED DIRECT SHEAR TEST
- PL Wc LL ATTERBERG LIMITS

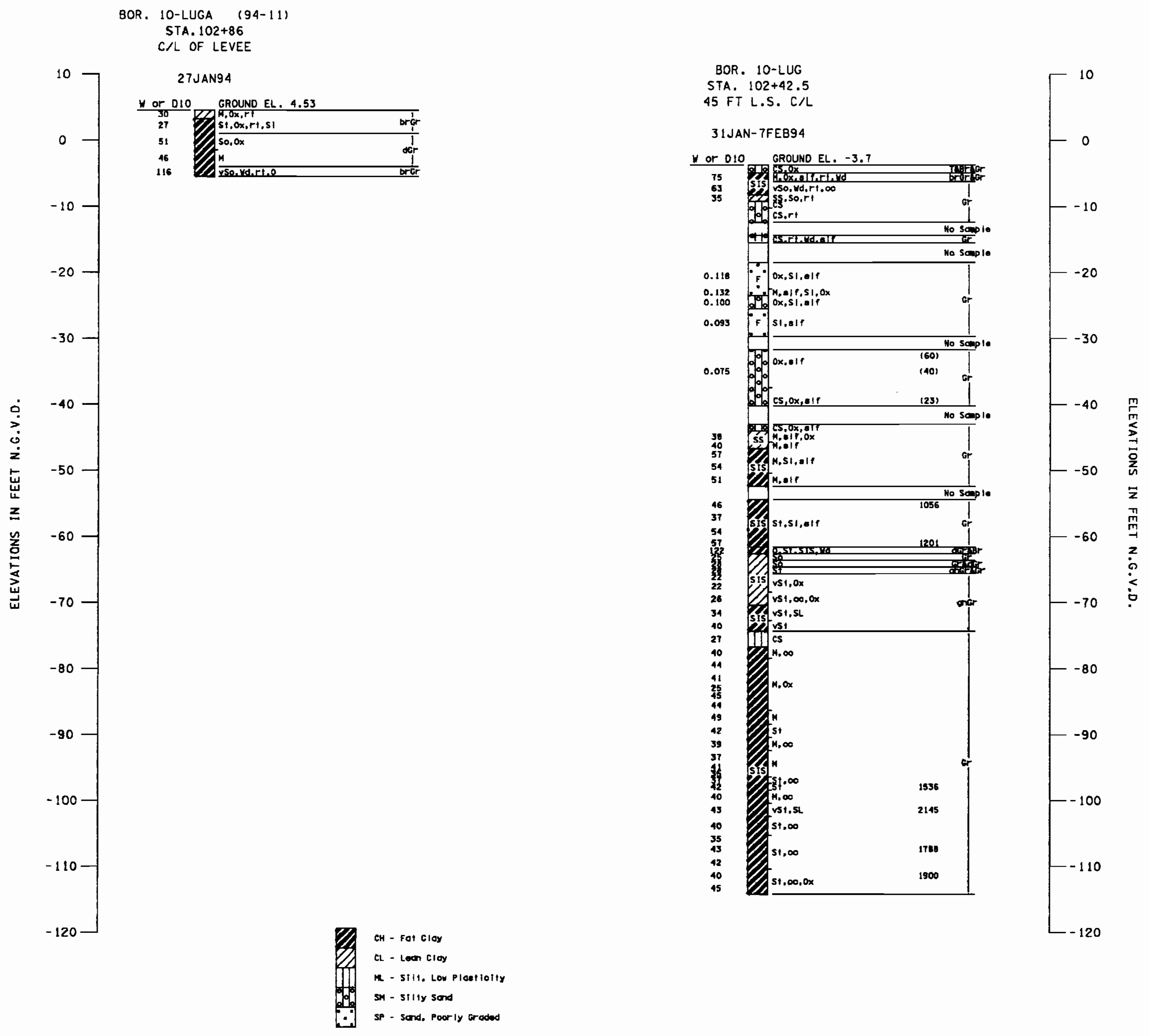
BORING WAS TAKEN WITH A 5 INCH DIAMETER SHELBY TUBE PISTON TYPE SAMPLER.
FOR SOIL BORING LEGEND SEE PLATE A.
FOR LOCATION OF BORING SEE PLATE 2
FOR DETAILED TEST DATA SEE APPENDIX

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
STA. 102+42.5

UNDISTURBED BORING 10-LUG
45 FT L.S. C/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 20:1	PLOT DATE: DEC 94	CADD FILE: LUGIO.DGN
DRAWN BY: WOODS	DATE: DEC 94		FILE NO. H-2-40511
CHECKED BY: RICHARDSON			



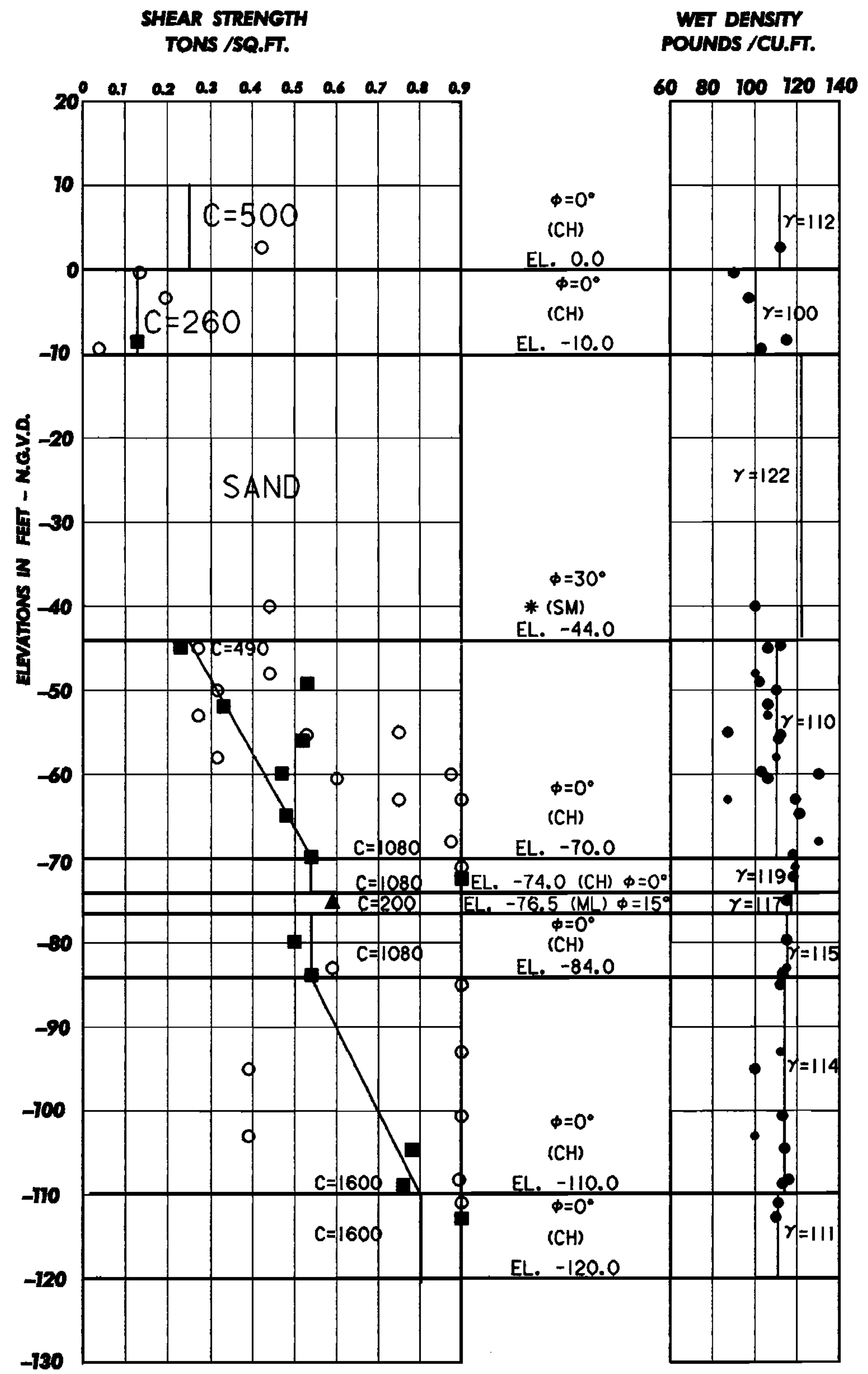
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
STA. 102+86 & STA. 102+42.5

GENERAL TYPE & UNDISTURBED BORING LOGS
10-LUGA & 10-LUG

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 20:1	PLOT DATE: DEC 94	CADD FILE: LUGIOAA.DGN
DRAWN BY: WOODS			FILE NO. H-2-40511
CHECKED BY: RICHARDSON	DATE: DEC 94		

PLATE 21



BORING LEGEND
SHEAR STRENGTHS
○ ■ 10-LUG PLATE 20
○ 56, 57 APPENDIX A
▲ 57

56 5" I.D. BORING
57 3" I.D. BORING
* FOR S SHEAR STRENGTHS SEE DM 19A LONDON AVENUE OUTFALL CANAL PLATE 61

○ UNCONFINED COMPRESSION TESTS
■ UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS
▲ UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST 1 PT EUSTIS ENGINEERING
SEE APPENDIX A FOR EUSTIS ENGINEERING BORINGS AND LABORATORY TESTS

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
LONDON AVE OUTFALL CANAL
SOIL DESIGN PARAMETERS
PUMPING STA NO. 4

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH DRAWN BY: WOODS CHECKED BY: RICHARDSON	PLOT SCALE: 20:1	PLOT DATE: DEC 94	CADD FILE: SHEAR04.DGN
	DATE: DEC 94		FILE NO. H-2-40511

Assumption : Line Source, Gravity Flow Partially Penetrating Infinite Line of Wells.

Aquifer SP/SM Ground Surface El. -11.0
Ground Surface at Excavation El. -14.33

Transformation of Anisotropic Soil Conditions to Isotropic Soil Conditions $K_H/K_V = 4$

$$K = \sqrt{K_H K_V} = \sqrt{(400\times10^{-4})(100\times10^{-4})} = .020 \text{ cm/sec} = .0394 \text{ ft/min}$$

COFFERDAM #1

Wellpoints Drawdown to El. -17.5, Canal Stage +5.7

Assuming an Equivalent Continuous Slot:

$H = 49.7$ ft. $v = 15$ ft. $h_o = 25$ ft.

$R = C(H-h_w)\sqrt{K} = 2(49.7-25)\sqrt{200} = 699$ ft. Fig 4-23 Eq. 1

Assuming spacing between wellpoints = a = 15':

$Q_p = [(0.73 + 0.27\frac{(H - h_o)}{H}] \frac{Ka}{2L}(H^2 - h_o^2)$ Fig. 4-3 Eq. 3

Where $L \geq 3H$ $L = 699$ ft. $\geq 3(49.7) = 149.1$ ft.

$Q_p = [(0.73 + 0.27\frac{(49.7 - 25)}{49.7}] \frac{.0394(15)}{2(699)} [(49.7)^2 - (25)^2] = .674$ cfm

Increase Q_p to 1.5 gpm from sectional flownet.

$h_D = h_o\ [\frac{1.48}{L}(H - h_o) + 1]$ Fig. 4-3 Eq. 4

$h_D = 25[\frac{1.48}{699}(49.7 - 25) + 1] = 26.31$ ft.

$h_D = 26.31$ ft. or El. -17.69

$H_w = H_e + H_s + H_f + H_v$ Fig. 4-25

Assuming a 5 ft. screen.

$H_w = .32$

$M = v + h_e - \Delta H_v - H_w = .32$

$M = 15 + 25 - 1.31 - .32 + 38.37$ ft.

Set header no higher than El. -5.6

Use F.S. = 1.35 pg 4-41

COFFERDAM #2

Wellpoints Drawdown to El. -14.0, Canal Stage +4.0

Assuming an Equivalent Continuous Slot:

$H = 48.0$ ft. $v = 15$ ft. $h_o = 28$ ft.

$R = C(H-h_w)\sqrt{K} = 2(48-28)\sqrt{200} = 566$ ft. Fig 4-23 Eq. 1

Assuming spacing between wellpoints = a = 15':

$Q_p = [(0.73 + 0.27\frac{(H - h_o)}{H}] \frac{Ka}{2L}(H^2 - h_o^2)$ Fig. 4-3 Eq. 3

Where $L \geq 3H$ $L = 566$ ft. $\geq 3(48.0) = 144$ ft.

$Q_p = [(0.73 + 0.27\frac{(48.0 - 28)}{48.0}] \frac{.0394(15)}{2(566)} [(48.0)^2 - (28)^2] = .669$ cfm

Increase Q_p to 1.5 gpm from sectional flownet.

$h_D = h_o\ [\frac{C_1 C_2}{L}(H - h_o) + 1]$ Fig. 4-3 Eq. 4

$h_D = 28[\frac{(1.0)(1.4)}{566}(48 - 28) + 1] = 29.39$ ft.

$h_D = 29.39$ ft. or El. -14.61

Head Loss in Well points:

$H_w = H_e + H_s + H_f + H_v$ Fig. 4-25

Assuming a 5 ft. screen.

$H_w = .32$

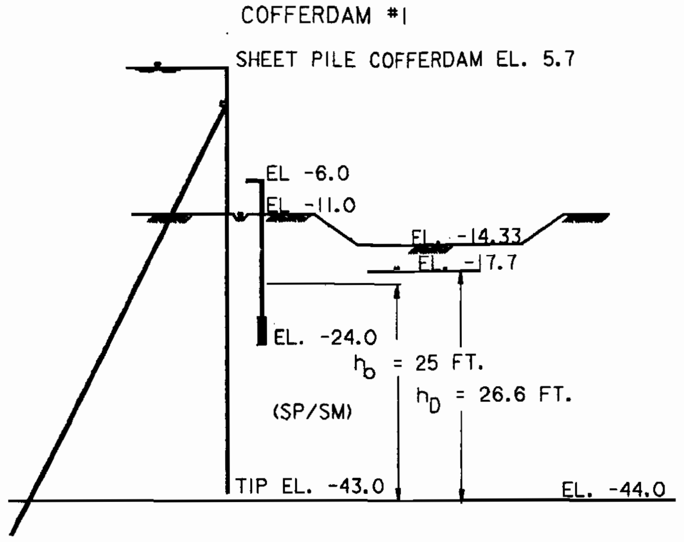
$M = v + h_e - \Delta H_v - H_w$

$M = 15 + 28 - 1.39 - .32 = 41.29$ ft.

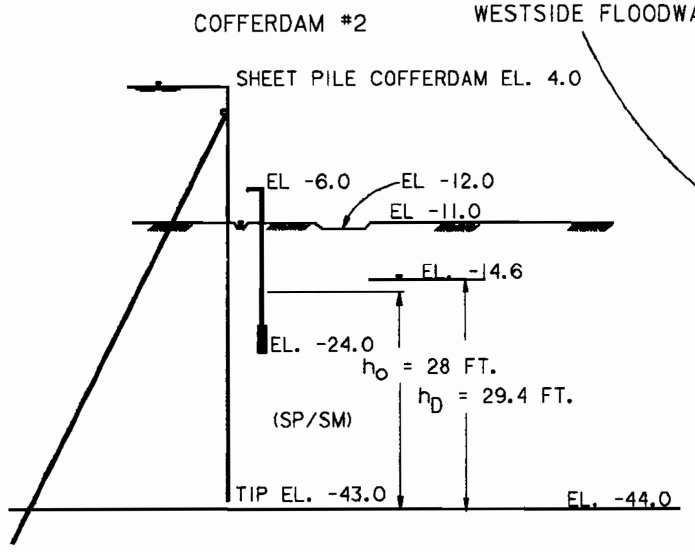
Set header no higher than El. -2.5

Use F.S. = 1.35 pg 4-41

Decrease spacing $a: \frac{15}{1.35} = 11$ ft.



SECTION A - A



SECTION B - B

Set Headers Elevation at El. -6.0

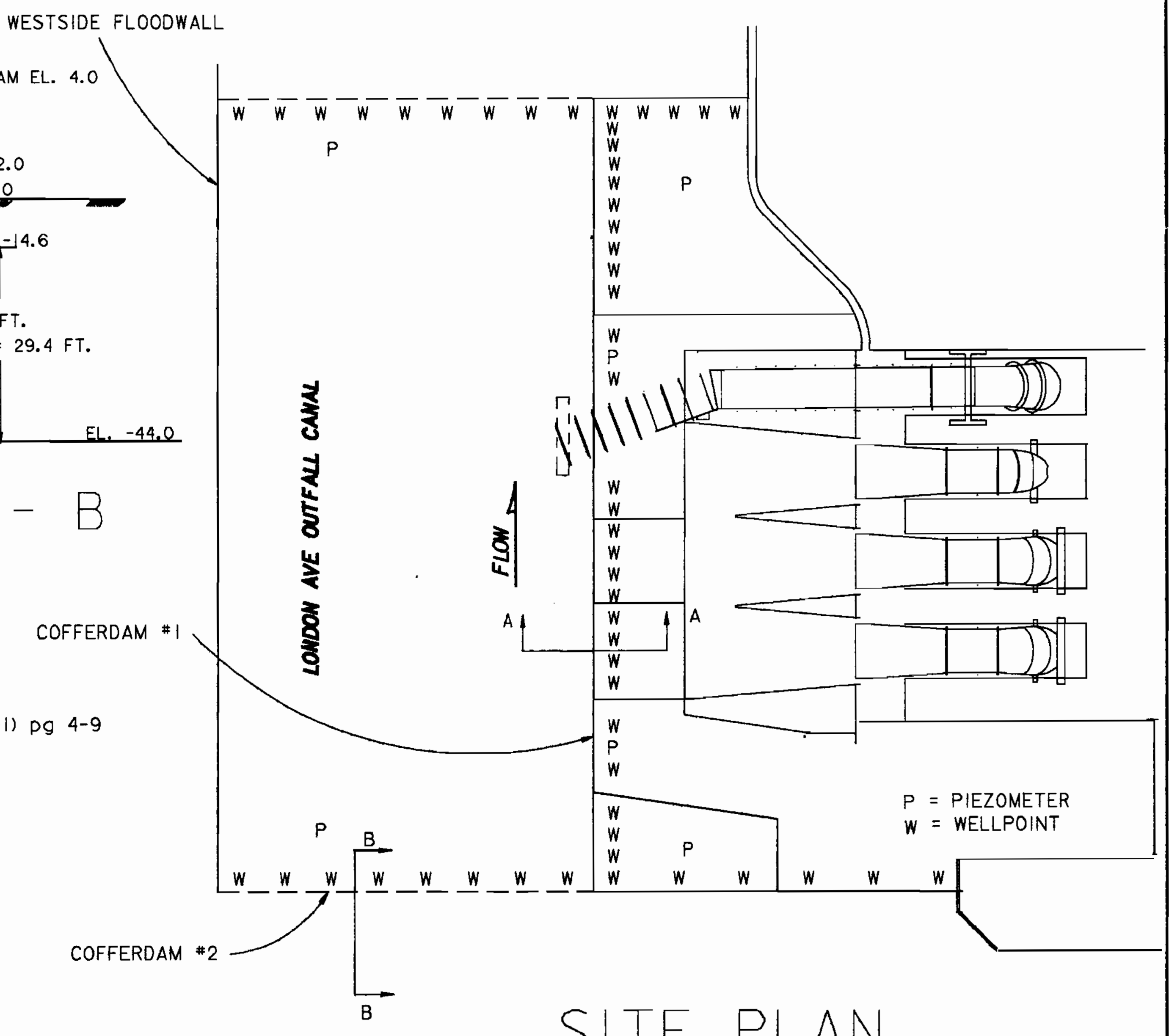
1.5 inch wellpoint. Tip El. -24.0

5 ft. screen with 5 inch filter from El. -12.0 to El.-24.0 .

Wetted Sreen: Limiting Flow $q_c = \frac{2\ r_w \sqrt{k}}{1.07} \times 7.48$ gpm per foot of screen Eq (4-1) pg 4-9

$q_c = \frac{2\ (3.25\ in/12in/ft)\sqrt{0.0394\ ft/min}}{1.07} \times 7.48$ gpm X 5 ft = 11.8 gpm

11.8 gpm > 1.5 gpm X 7.48 gpm = 5.0 gpm



SITE PLAN

Reference TM - 5 - 818 - 5 Nov. 1983.

* K_H based on D_{10} from graph plate 98 Design Memorandum No. 19 General Design London Ave. Outfall Canal.

CONSTRUCTION PROCEDURE COFFERDAM #1:

(1) Drive Sheet pile.
(2) Sump pump out water to El. -11.0 (F..S. = i_{cr}/i_e > 4.0 from flow net and Harr's Seepage Analyses)
(3) Place wellpoints with headers at El. -6.0.
(4) Excavate to El. -14.33.

CONSTRUCTION PROCEDURE COFFERDAM #2:

(1) Drive Sheet pile.
(2) Sump pump out water to El. -11.0 (F..S. = i_{cr}/i_e > 4.0 from flow net and Harr's Seepage Analyses)
(3) Place wellpoints with headers at El. -6.0.
(4) Excavate to El. -12.0.

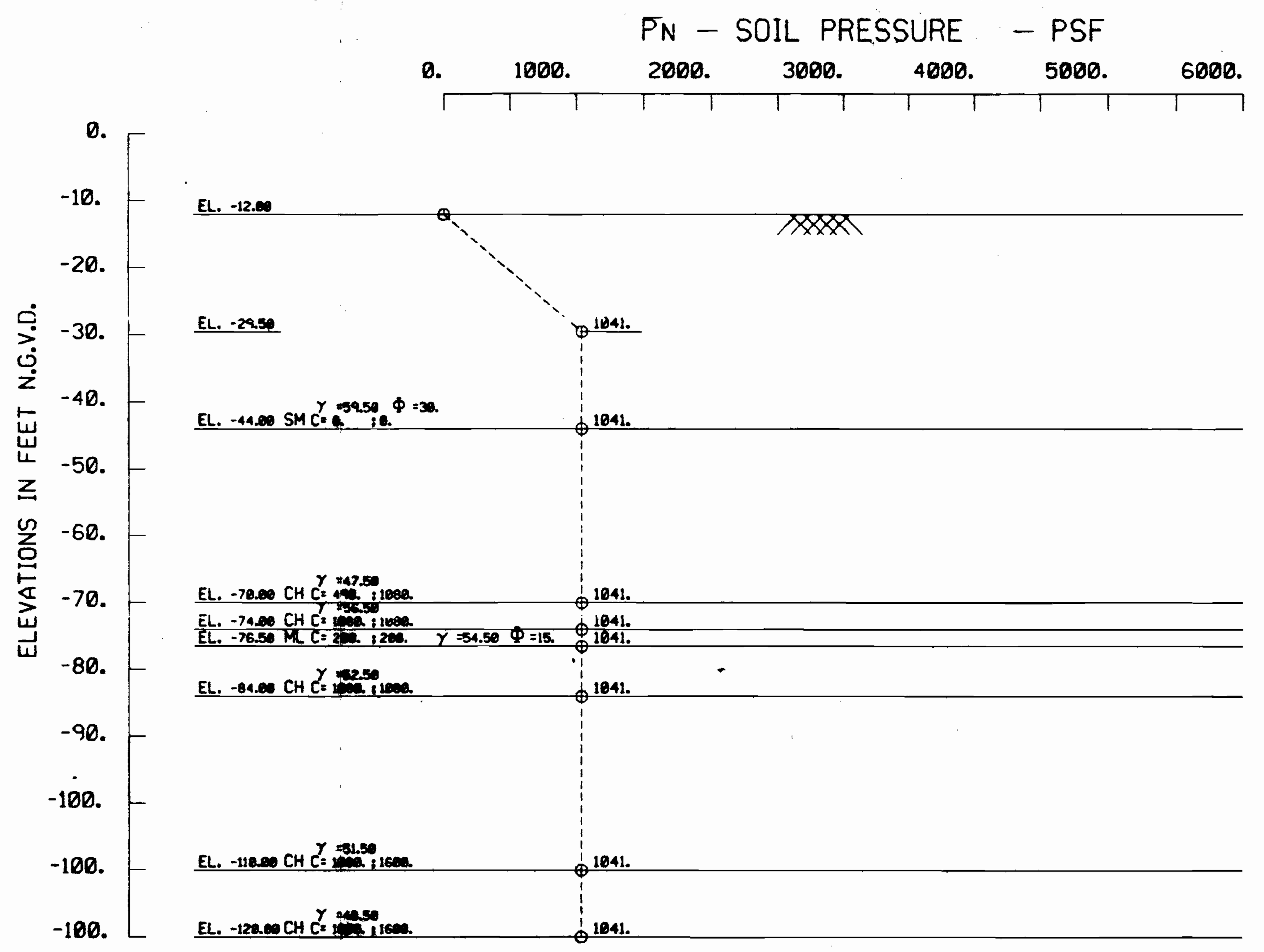
LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

DEWATERING SYSTEM

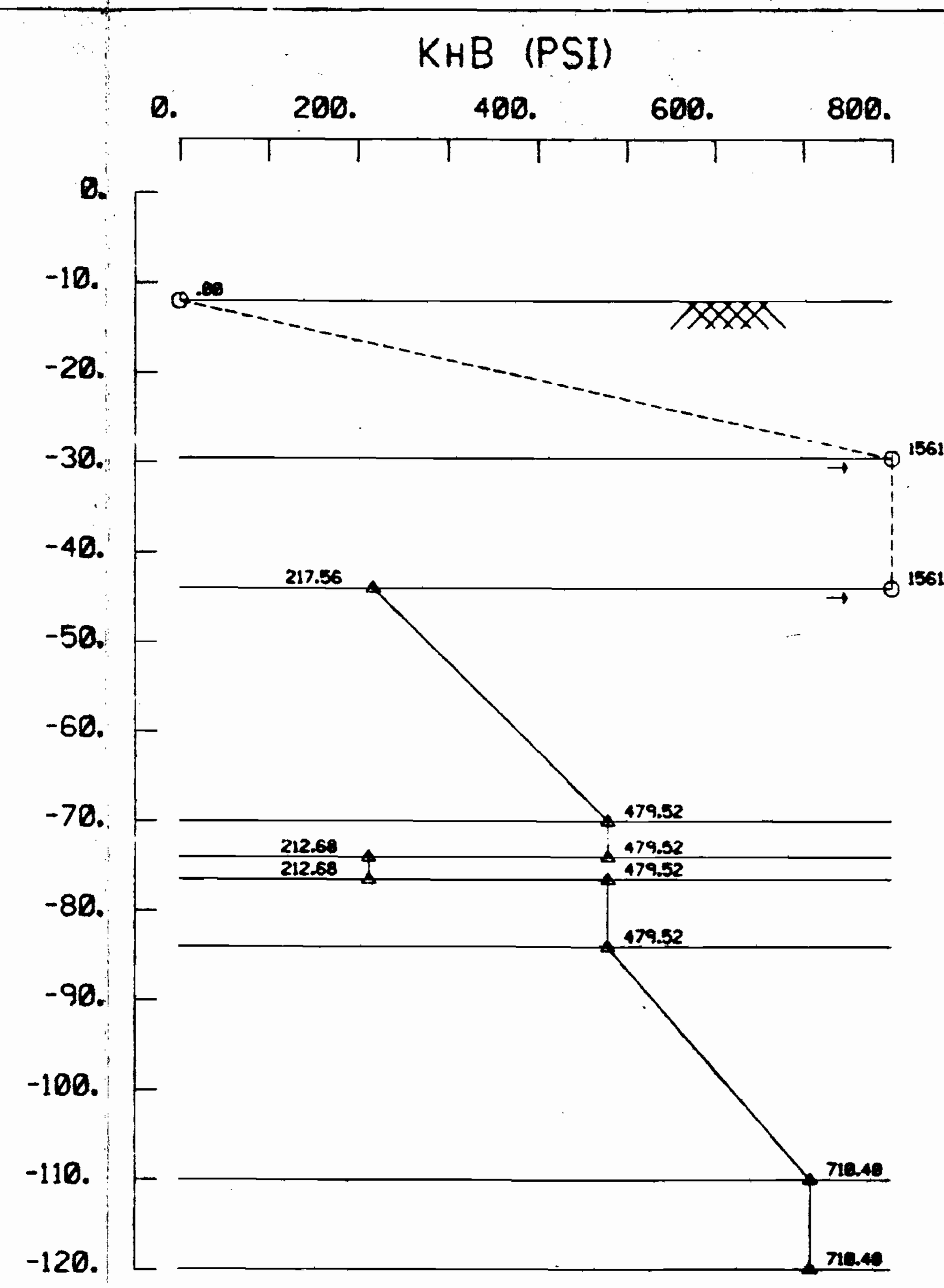
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 2 : 1	PLOT DATE: DEC 1994	CADD FILE: ROT
DRAWN BY: WOODS			FILE NO.
CHECKED BY: RICHARDSON	DATE: DEC 1994		H-2-40511

PLATE 23



S-CASE
CH,CL- $\Phi = 23°$
ML,SM,SP- $\Phi = 30°$

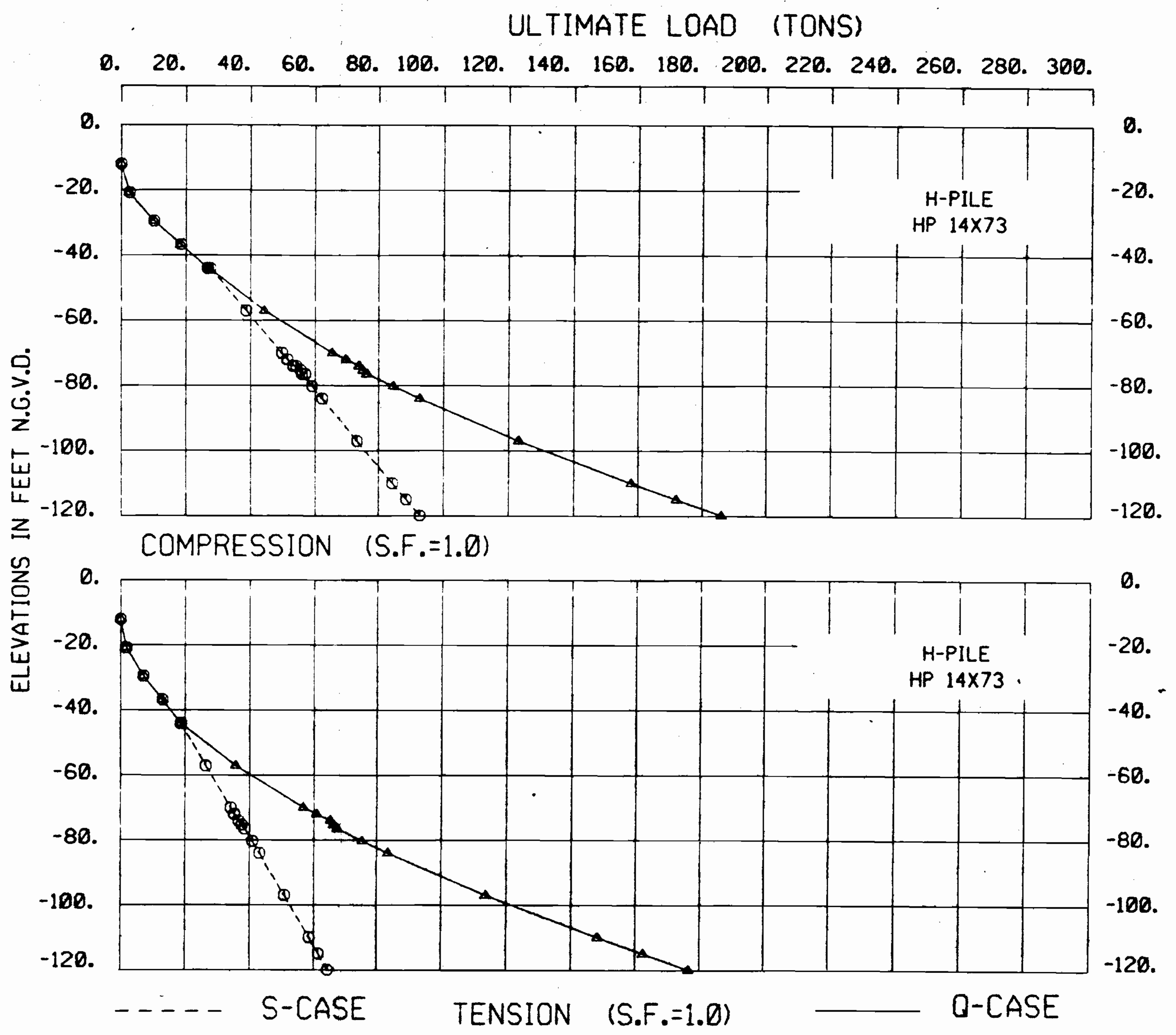


TYPICAL SOIL PROFILE

SOIL STRATIFICATION IS BASED ON GEOLOGIC PROFILE.
SHEAR STRENGTH AND WET DENSITIES, SEE PLATE 22

D	PILE SPACING IN DIRECTION OF LOADING
1.00	8B
.85	7B
.70	6B
.55	5B
.40	4B
.25	3B

C	LOADING CONDITION
1.00	INITIAL LOADING
0.30	CYCLIC LOADING



NOTES:

1. ALLOWABLE Q-CASE CAPACITIES MUST BE DETERMINED INCORPORATING F.S.=2.0 WITH PILE LOAD TEST OR F.S.=3.0 WITHOUT PILE LOAD TEST
2. ALLOWABLE S-CASE CAPACITIES MUST BE DETERMINED INCORPORATING F.S.=2.0 FOR DEAD LOAD ONLY AND F.S.=1.5 FOR TOTAL LOAD WITH PILE TEST OR F.S.=3.0 DEAD LOAD ONLY AND F.S.=2.25 TOTAL LOAD WITHOUT PILE TEST.
3. PILE CAPACITIES ARE BASED ON NO JETTING AND NO PREDRILLING.
4. THE SAME PILE HAMMER MUST BE USED FOR BOTH THE TEST PILES AND THE SERVICE PILES.
5. PILE CAPACITIES ARE BASED ON PILE DRIVING WITH AN IMPACT HAMMER.
6. PILE CAPACITIES MUST BE REDUCED FOR PILE DRIVING WITH A VIBRATORY HAMMER.

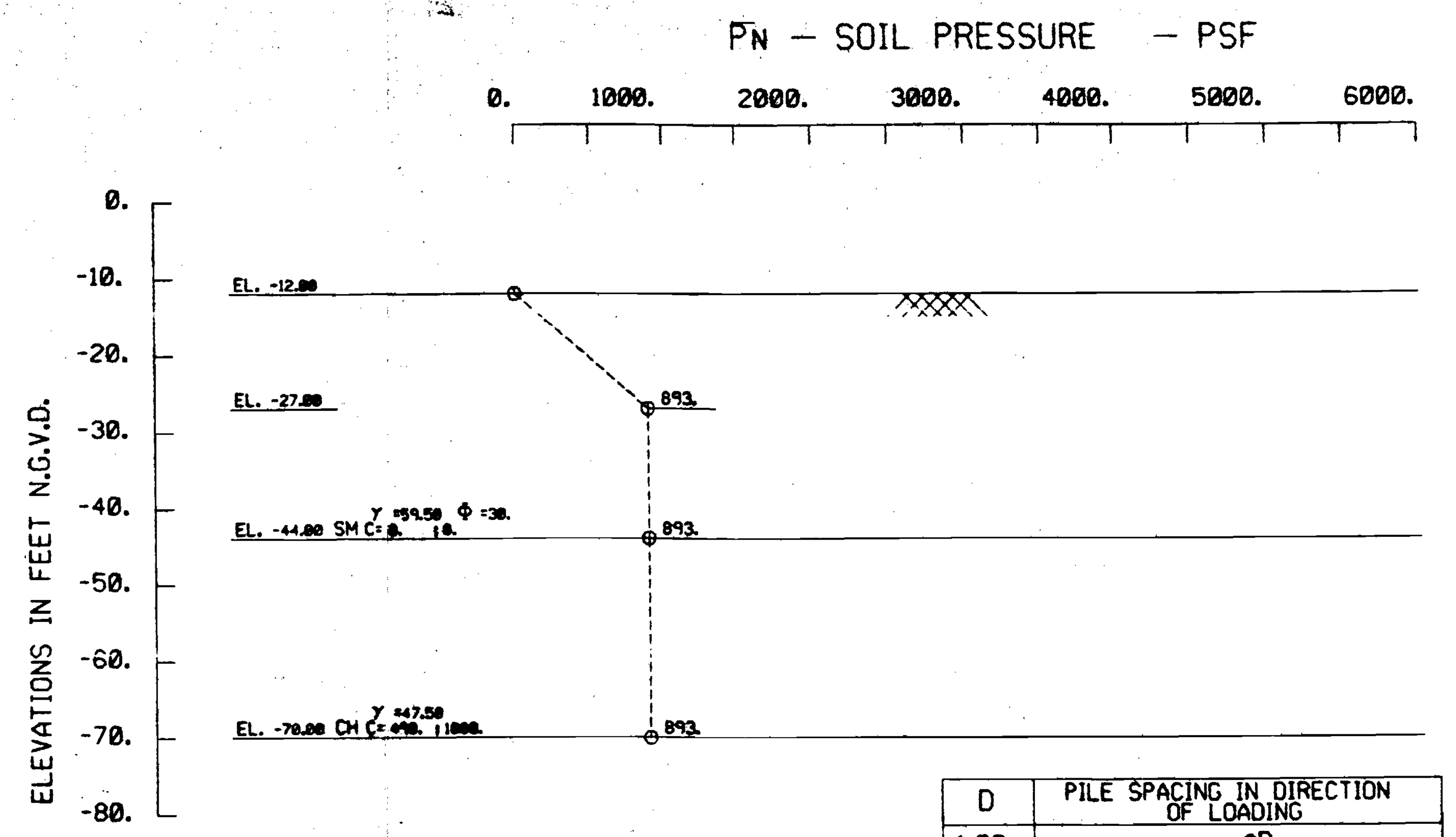
THE FACTOR SHOWN, (MODULUS OF HORIZONTAL SUBGRADE Kh, TIMES THE PILE WIDTH IN INCHES (B), MEASURED AT RIGHT ANGLES TO THE DIRECTION OF DISPLACEMENT) MUST BE MODIFIED BY A REDUCTION FACTOR FOR THE EFFECT OF GROUP ACTION (D) AND A REDUCTION FACTOR FOR CYCLIC LOADING (C) EX: $K_h = \frac{0.2222 q_u (C)(D)}{(B)}$

SEE PLATE 25 FOR KHB NOTES.

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
LONDON AVE OUTFALL CANAL
PILE CAPACITY CURVES
PUMPING STATION NO. 4 - HP 14X73 STEEL PILES

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH | DRAWN BY: WOODS | CHECKED BY: RICHARDSON | PLOT SCALE: 1:1 | PLOT DATE: MAY 95 | DATE: DEC 94 | FILE NO. H-2-40511

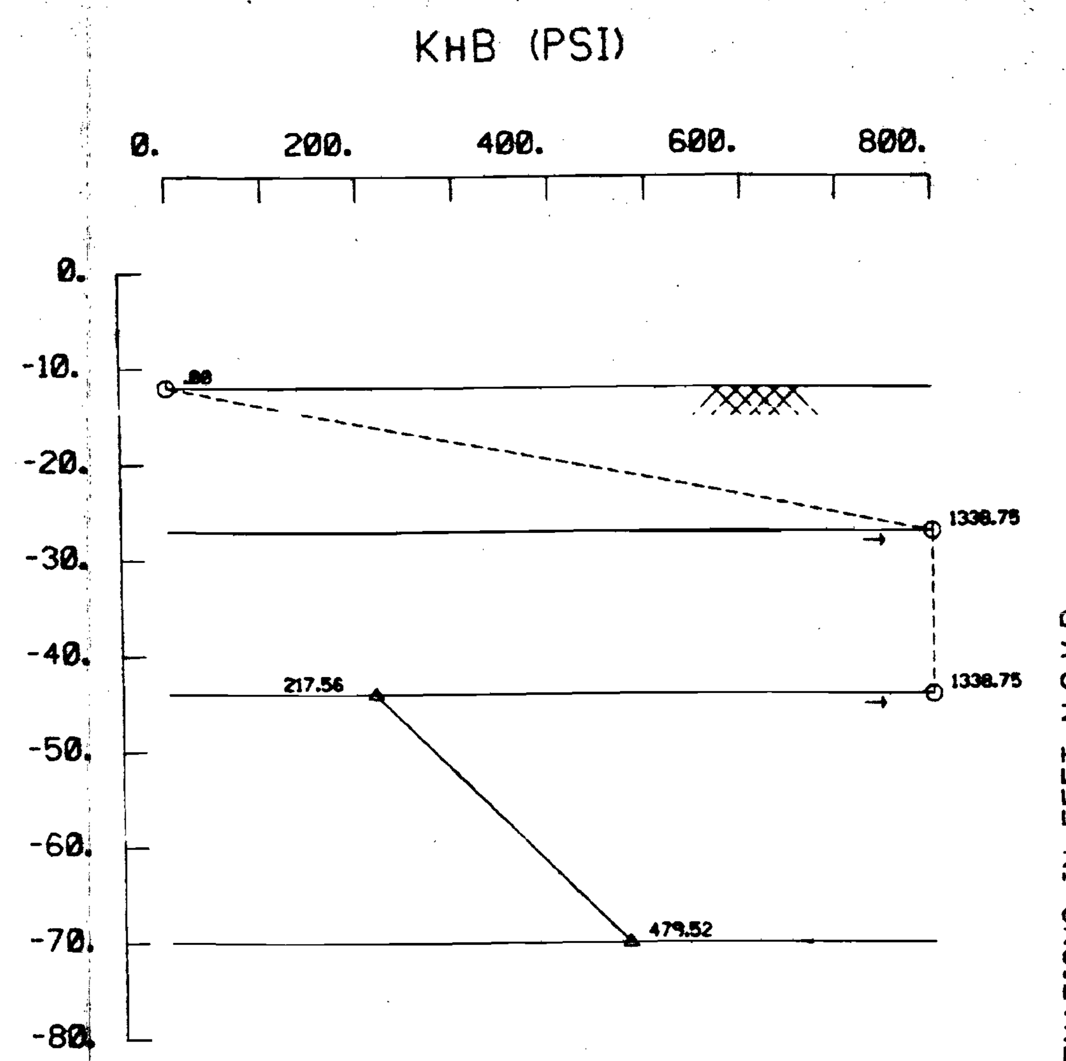


S-CASE
CH,CL- Φ=23°
SM,SP- Φ=30°

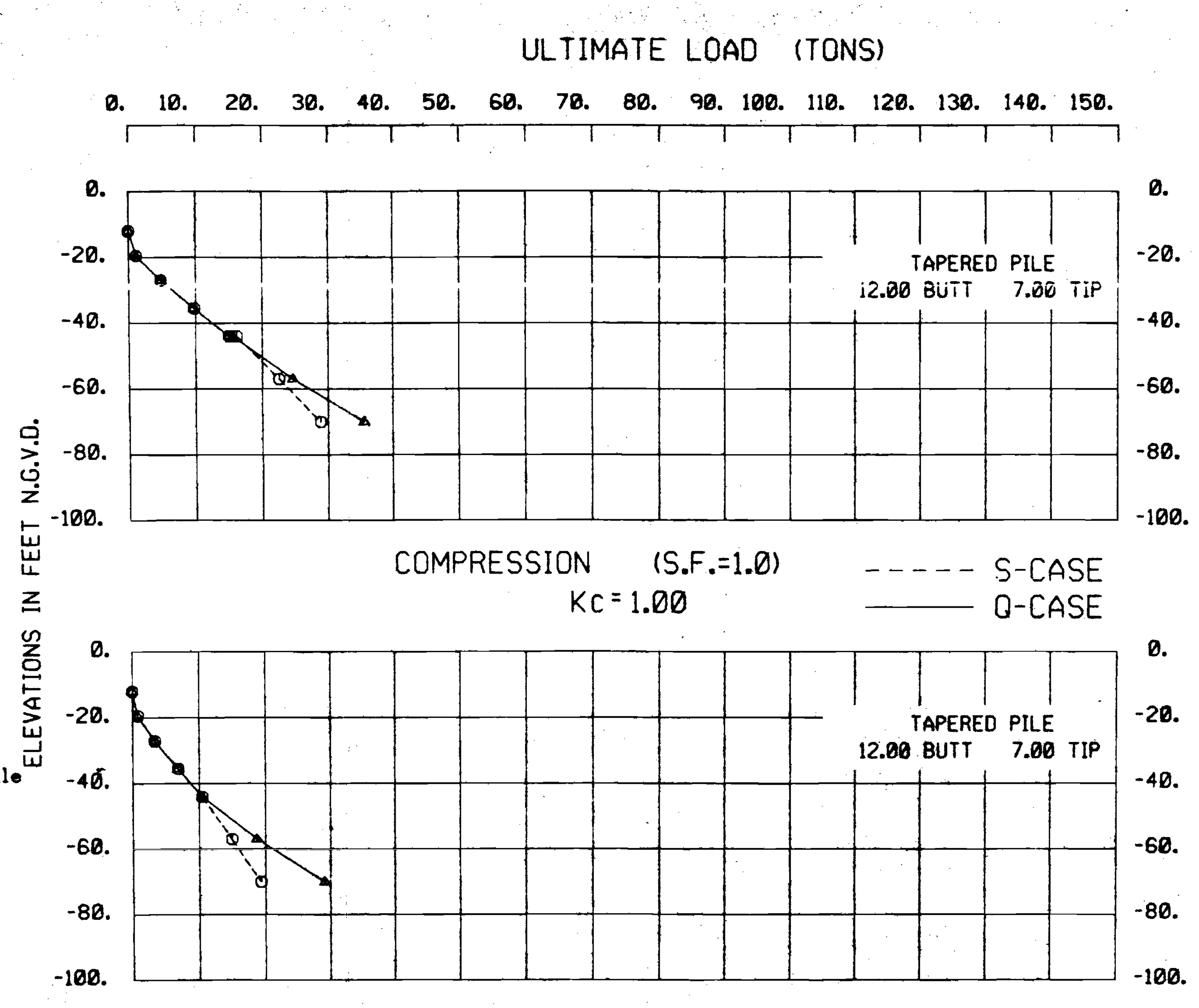
TYPICAL SOIL PROFILE

SOIL STRATIFICATION IS BASED ON GEOLOGIC PROFILE.
SHEAR STRENGTH AND WET DENSITIES SEE PLATE 22.

D	PILE SPACING IN DIRECTION OF LOADING
1.00	8B
.85	7B
.70	6B
.55	5B
.40	4B
.25	3B
C	LOADING CONDITION
1.00	INITIAL LOADING
0.30	CYCLIC LOADING



NOTES: KH = ϵK1/B=(0.2222qu/BXCXD) COHESIVE
ϵ = 0.4 = Factor of material properties of soil and pile
k1 = Modulus of subgrade reaction for test plate (pci)
B1 = Width or diameter of test plate (in)
K1 = k1B1 = 80qu(pcf) = 0.5556qu(psi)
qu = 2*c = Unconfined compressive strength (psf)
C = Reduction for cyclic loading-not applicable
D = Group effect reduction factor
B = Width of pile measured at right angles to the direction of displacement (in)
KH = (nhXZ/BXCXD) COHESIONLESS
nh = Coeficient of horizontal subgrade reaction (pci)
Z = Depth below equivalent ground surface (in)



TENSION (S.F.=1.0)
KT = 0.70

THE FACTOR SHOWN (MODULUS OF HORIZONTAL SUBGRADE Kh, TIMES THE PILE WIDTH IN INCHES (B), MEASURED AT RIGHT ANGLES TO THE DIRECTION OF DISPLACEMENT) MUST BE MODIFIED BY A REDUCTION FACTOR FOR THE EFFECT OF GROUP ACTION (D) AND A REDUCTION FACTOR FOR CYCLIC LOADING (C) EX: $K_h = \frac{0.2222qu/(CXD)}{(B)}$

NOTES:

1. ALLOWABLE Q-CASE CAPACITIES MUST BE DETERMINED INCORPORATING F.S.=2.0 WITH PILE TEST OR F.S.=3.0 WITHOUT PILE TEST.
2. ALLOWABLE S-CASE CAPACITIES MUST BE DETERMINED INCORPORATING F.S.=2.0 FOR DEAD LOAD ONLY AND F.S.=1.5 FOR TOTAL LOAD WITH PILE TEST OR F.S.=3.0 DEAD LOAD ONLY AND F.S.=2.25 TOTAL LOAD WITHOUT PILE TEST.
3. PILE CAPACITIES ARE BASED ON NO JETTING AND NO PREDRILLING.

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A, GENERAL DESIGN
SUPPLEMENT NO. 1
LONDON AVE OUTFALL CANAL

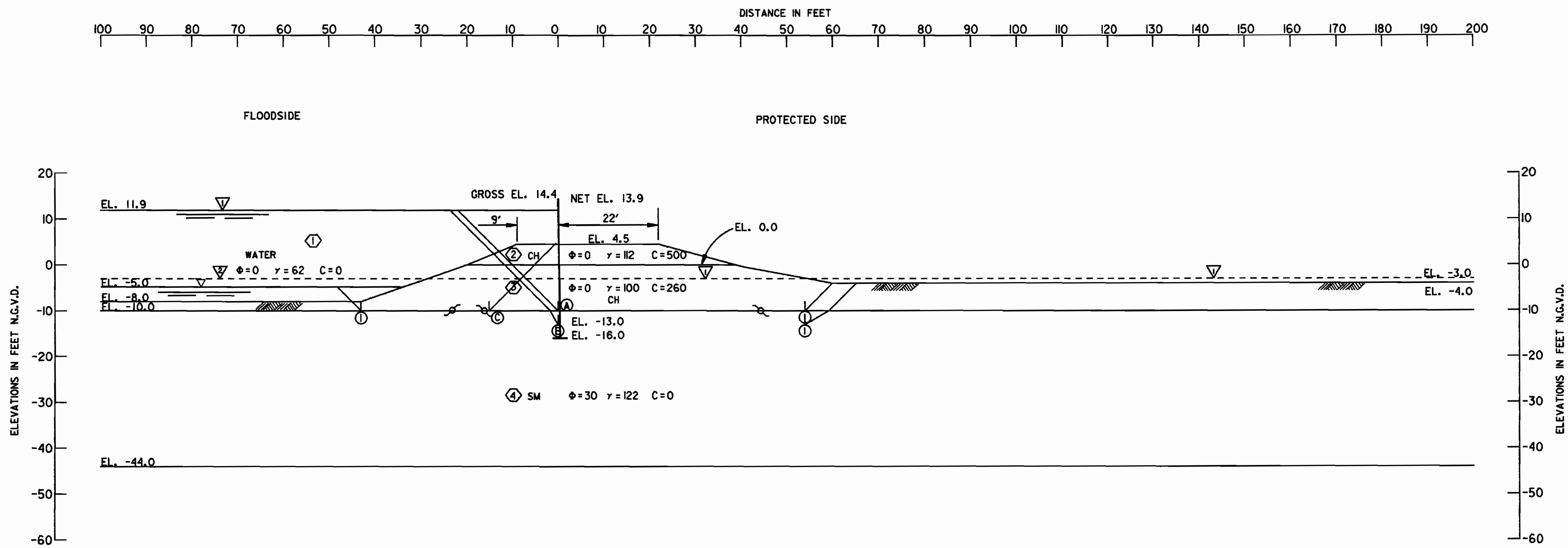
PILE CAPACITY CURVES

PUMPING STATION NO. 4 - CLASS B TIMBER PILES

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH | DRAWN BY: WOODS | CHECKED BY: RICHARDSON | PLOT SCALE: 30:1 | PLOT DATE: MAY 95 | DATE: DEC 94 | FILE NO. H-2-40511

ENCL 3 REVISED MAY 95 PLATE 25



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
Ⓐ ①	-10.0	8104	13547	3120	19170	2131	24771	17039	1.45
Ⓑ ①	-13.0	8898	30719	5006	25410	4343	44623	21067	2.12
Ⓒ ①	-10.0	9700	3963	1040	10345	855	14703	9490	1.55

NOTES

⬡ -- STRATUM NUMBER
○ -- WEDGE NUMBER
-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES.

∇ ∇ PH LINE IN STRATUM ④ PROTECTED SIDE ANALYSIS
∇ ∇ PH LINE IN STRATUM ④ FLOOD SIDE ANALYSIS

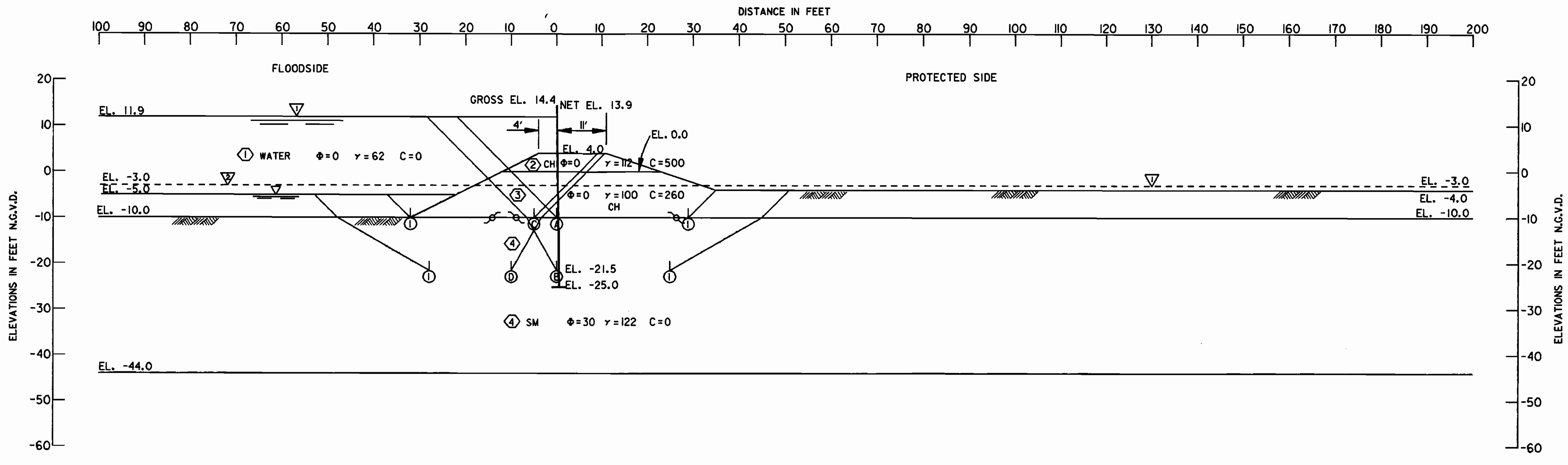
LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

STABILITY ANALYSIS

STA. 0+00 SUB B/L TO STA. 0+76.89 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 20:1	PLOT DATE: DEC 94	CADD FILE: REST
DRAWN BY: WOODS			FILE NO. H-2-40511
CHECKED BY: RICHARDSON	DATE: DEC 94		



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	SAFETY
Ⓐ ①	-10.0	5869	7473	3120	18308	2400	16462	15908	1.03
Ⓑ ①	-21.5	11205	22369	16649	47149	17728	50223	29421	1.71
Ⓒ ①	-10.0	9200	3730	0	10346	780	12930	9566	1.35
Ⓓ ①	-21.5	18402	9648	5155	33342	12522	33205	20820	1.59

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ∽ -- CROSSOVER POINT
- φ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

GENERAL NOTES:

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTH, AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS. SEE BORING DATA PLATES.

∇ ∇ PH LINE IN STRATUM ④ PROTECTED SIDE ANALYSIS

∇ ∇ PH LINE IN STRATUM ④ FLOODSIDE ANALYSIS

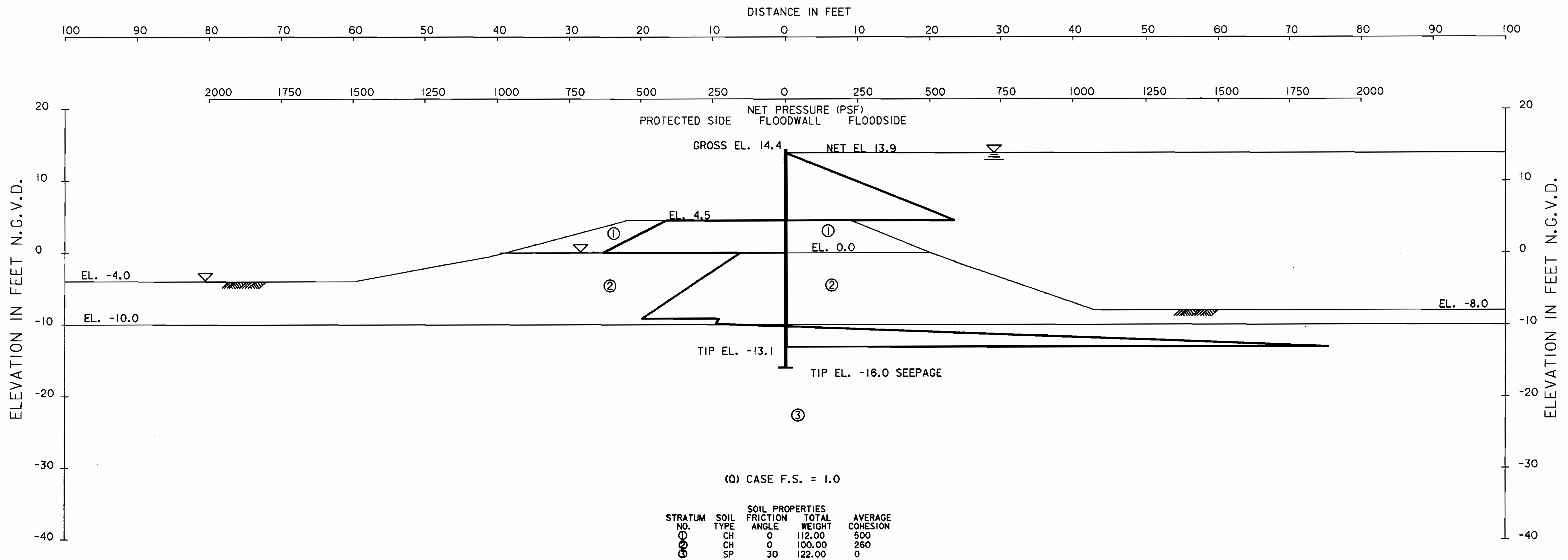
LAKE PONTCHARTRAIN, LA AND VIVINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

STABILITY ANALYSIS

STA. 2+47.39 SUB B/L TO STA. 3+00.58 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH DRAWN BY: WOODS CHECKED BY: RICHARDSON	PLOT SCALE: 20:1	PLOT DATE: DEC 94	CADD FILE: SIWD
	DATE: DEC 94		FILE NO. H-2-40511



(Q) CASE F.S. = 1.0

STRATUM NO.	SOIL TYPE	SOIL PROPERTIES FRICTION ANGLE	TOTAL WEIGHT	AVERAGE COHESION
①	CH	0	112.00	500
②	CH	0	100.00	260
③	SP	30	122.00	0

SEE PLATE 22 FOR SOIL DESIGN PARAMETERS.

SEE PLATE 26 FOR STABILITY ANALYSIS.

ELEVATION	PRESSURE
13.90	.00
4.50	587.50
4.50	-412.50
.00	-635.25
.00	-155.25
-3.57	-289.26
-3.57	-289.26
-4.00	-305.25
-4.00	-305.25
-5.00	-342.75
-5.00	-342.75
-9.15	-498.43
-9.15	-229.02
-9.87	-239.31
-13.06	1886.28
-13.06	.00

SEEPAGE CALCULATED BASED ON LANE'S WEIGHTED CREEP RATIO.
ASSUME UPPER LAYER IS A SAND THEREFORE LWCR $\geq$ 4. TRANSFORMATION OF CLAY LAYER TO SAND LAYER = 4/1.8.

$$LWCR = \frac{\text{WEIGHTED CREEP DISTANCE}}{\Delta\ \text{HEAD}}$$

MINIMUM TIP ELEVATION FOR SEEPAGE:

$$4 \geq \frac{103'/3 + 2d + 4/1.8(6' + 2')}{15.9'} =$$

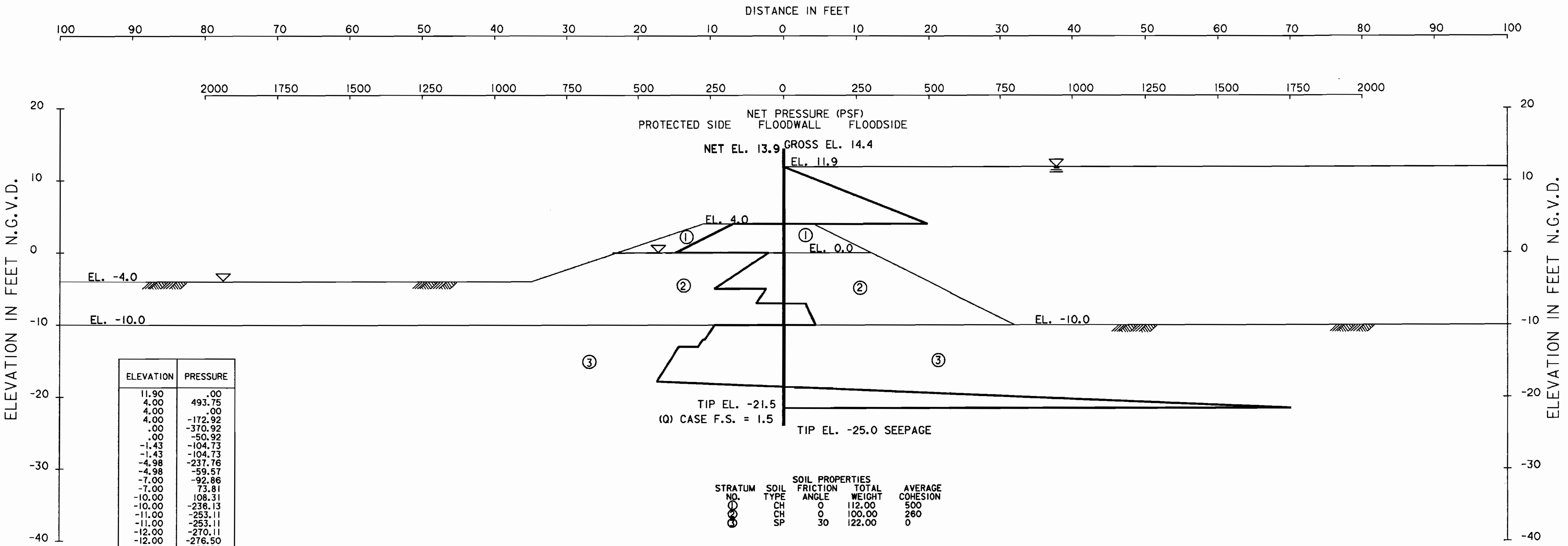
d = 5.7 EL. -16.0

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH
I-WALL ANALYSIS
STA. 0+00 SUB B/L TO STA. 0+76.89 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 10:1	PLOT DATE: DEC 94	CADD FILE: SOUTH.DGN
DRAWN BY: WOODS			FILE NO. H-2-40511
CHECKED BY: RICHARDSON	DATE: DEC 94		

PLATE 28



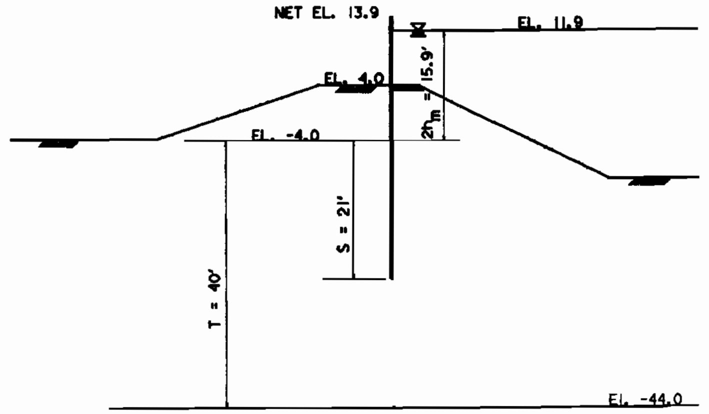
ELEVATION	PRESSURE
11.90	.00
4.00	493.75
4.00	.00
4.00	-172.92
.00	-370.92
.00	-50.92
-1.43	-104.73
-1.43	-104.73
-4.98	-237.76
-4.98	-59.57
-7.00	-92.86
-7.00	73.81
-10.00	108.31
-10.00	-238.13
-11.00	-253.11
-11.00	-253.11
-12.00	-270.11
-12.00	-276.50
-13.00	-293.38
-13.00	-360.24
-17.81	-435.77
-21.52	1751.71
-21.52	.00

SOIL PROPERTIES

STRATUM NO.	SOIL TYPE	FRICTION ANGLE	TOTAL WEIGHT	AVERAGE COHESION
1	CH	0	112.00	500
2	CH	0	100.00	260
3	SP	30	122.00	0

SEE PLATE 22 FOR SOIL DESIGN PARAMETERS.

SEE PLATE 26 FOR STABILITY ANALYSIS.



SEEPAGE CALCULATED BASED ON HARR'S METHOD
ASSUME SAND EXTENDS TO THE SURFACE AND
HORIZONTAL GROUND SURFACE AT EL. -4.0

S = 21'   T = 40' $2h_m$ = 15.9'

S/T = 21'/40' = .52

$\frac{I_e S}{h_m}$ = 0.58 for S/T = 0.51 FROM HARR'S CHART

$I_e = \frac{0.58\ h_m}{S} = \frac{0.59\ (7.95')}{21'} = 0.22$

I_{cr} = 59.5 PCF/62.5 PCF = 0.95

F.S. = $\frac{I_{cr}}{I_e} = \frac{0.95}{0.22} = 4.2$

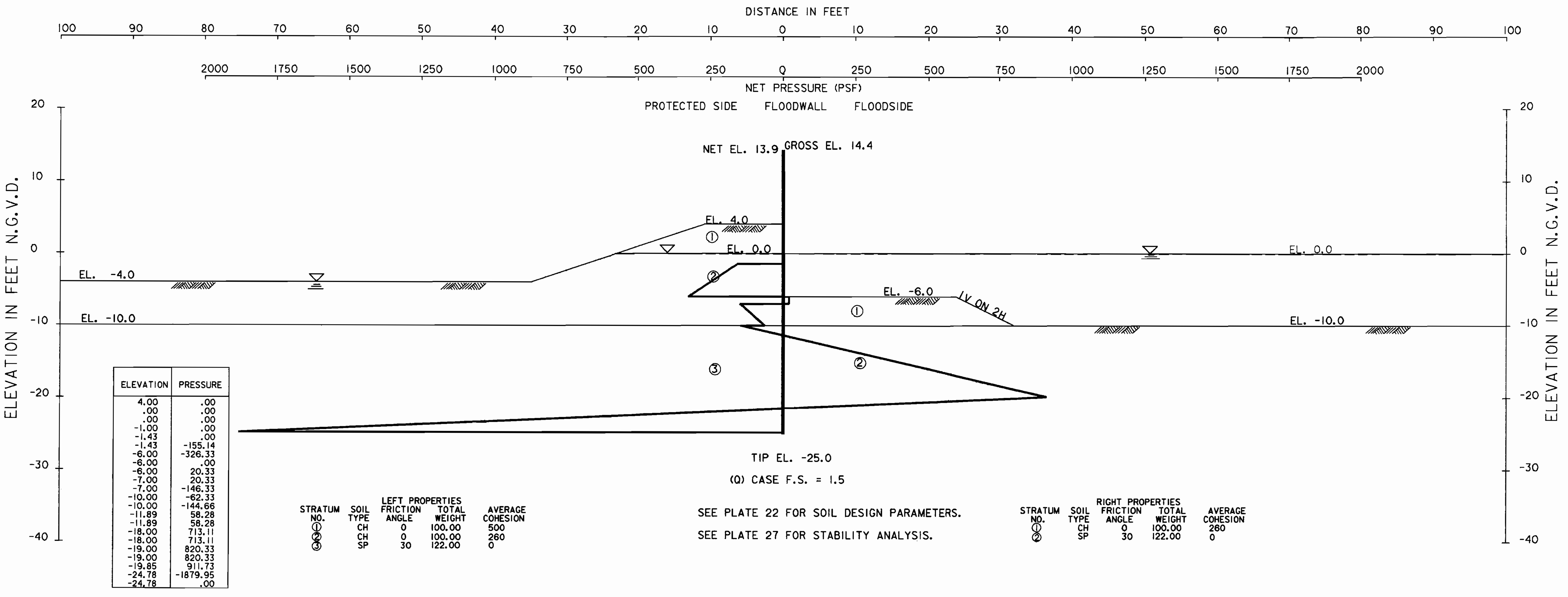
4.2 $\geq$ 4.0 FOR (SM) AND (SP) OK

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH
I-WALL ANALYSIS
STA. 2+73.08 SUB B/L TO STA. 3+00.58 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH DRAWN BY: WOODS CHECKED BY: RICHARDSON	PLOT SCALE: 10:1	PLOT DATE: DEC 94	CADD FILE: IWALL.DGN FILE NO. H-2-40511
	DATE: DEC 94		

PLATE 29



ELEVATION	PRESSURE
4.00	.00
.00	.00
.00	.00
-1.00	.00
-1.43	.00
-1.43	-155.14
-6.00	-326.33
-6.00	.00
-6.00	20.33
-7.00	20.33
-7.00	-146.33
-10.00	-62.33
-10.00	-144.66
-11.89	58.28
-11.89	58.28
-18.00	713.11
-18.00	713.11
-19.00	820.33
-19.00	820.33
-19.85	911.73
-24.78	-1879.95
-24.78	.00

TIP EL. -25.0

(Q) CASE F.S. = 1.5

SEE PLATE 22 FOR SOIL DESIGN PARAMETERS.

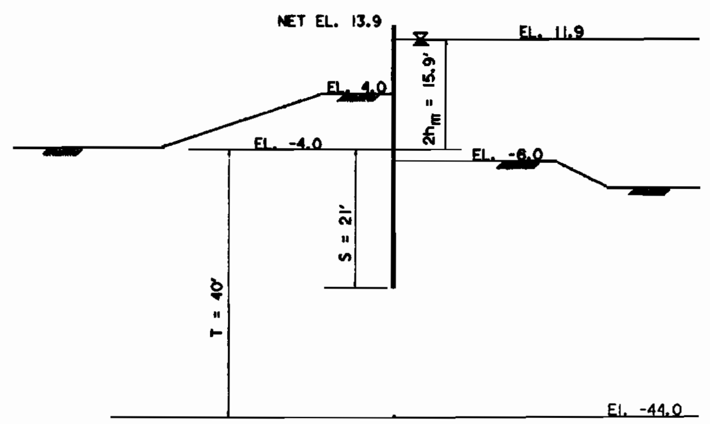
SEE PLATE 27 FOR STABILITY ANALYSIS.

LEFT PROPERTIES

STRATUM NO.	SOIL TYPE	FRICTION ANGLE	TOTAL WEIGHT	AVERAGE COHESION
1	CH	0	100.00	500
2	CH	0	100.00	260
3	SP	30	122.00	0

RIGHT PROPERTIES

STRATUM NO.	SOIL TYPE	FRICTION ANGLE	TOTAL WEIGHT	AVERAGE COHESION
1	CH	0	100.00	260
2	SP	30	122.00	0



SEEPAGE CALCULATED BASED ON HARR'S METHOD
ASSUME SAND EXTENDS TO THE SURFACE AND
HORIZONTAL GROUND SURFACE AT EL. -4.0

$S = 21'$ $T = 40'$ $2h_m = 15.9'$

$S/T = 21'/40' = .52$

$\frac{I_e S}{h_m} = 0.58$ for $S/T = 0.52$ FROM HARR'S CHART

$I_e = \frac{0.58\ h_m}{S} = \frac{0.58\ (7.95')}{21'} = 0.22$

$I_{cr} = 59.5\ PCF/62.5\ PCF = 0.95$

$F.S. = \frac{I_{cr}}{I_e} = \frac{0.95}{0.22} = 4.3$

$4.3 \geq 4.0$ FOR (SM) AND (SP) OK

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

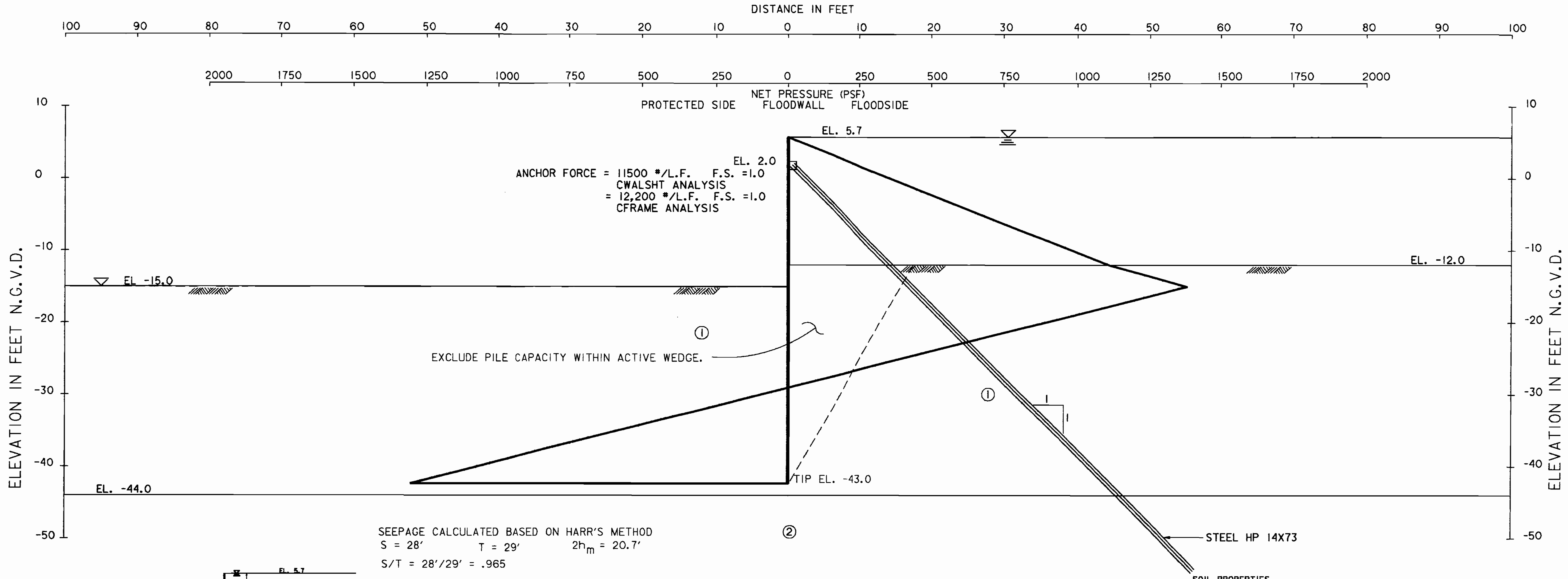
I - WALL ANALYSIS

STA. 2+47.39 SUB B/L TO STA. 2+73.08 SUB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

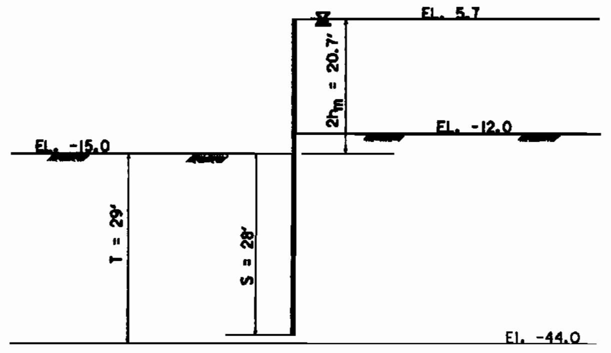
DESIGNED BY: VOJKOVICH | DRAWN BY: WOODS | CHECKED BY: RICHARDSON | PLOT SCALE: 10:1 | PLOT DATE: DEC 94 | DATE: DEC 94 | CADD FILE: BULLDGH | FILE NO. H-2-40511

PLATE 30



ELEVATION	PRESSURE
5.70	.00
-12.00	1106.25
-12.00	1106.25
-15.00	1377.90
-15.00	1377.90
-42.35	-1306.30
-42.35	.00

STRATUM NO.	SOIL TYPE	FRICTION ANGLE	TOTAL WEIGHT	AVERAGE COHESION
		SOIL PROPERTIES		
①	SP	30	122.00	0
②	CH	0	110.00	785



SEEPAGE CALCULATED BASED ON HARR'S METHOD

S = 28' T = 29' $2h_m$ = 20.7'

S/T = 28'/29' = .965

$\frac{I_e S}{h_m}$ = 0.33 for S/T = 0.965 FROM HARR'S CHART

$I_e = \frac{0.33\ h_m}{S} = \frac{0.33\ (20.7')}{28'} = 0.24$

I_{cr} = 59.5 PCF/62.5 PCF = 0.95

$F.S. = \frac{I_{cr}}{I_e} = \frac{0.95}{0.24} = 4.0$

4.0 $\geq$ 4.0 FOR (SM) AND (SP) OK

(Q) CASE F.S. = 1.5

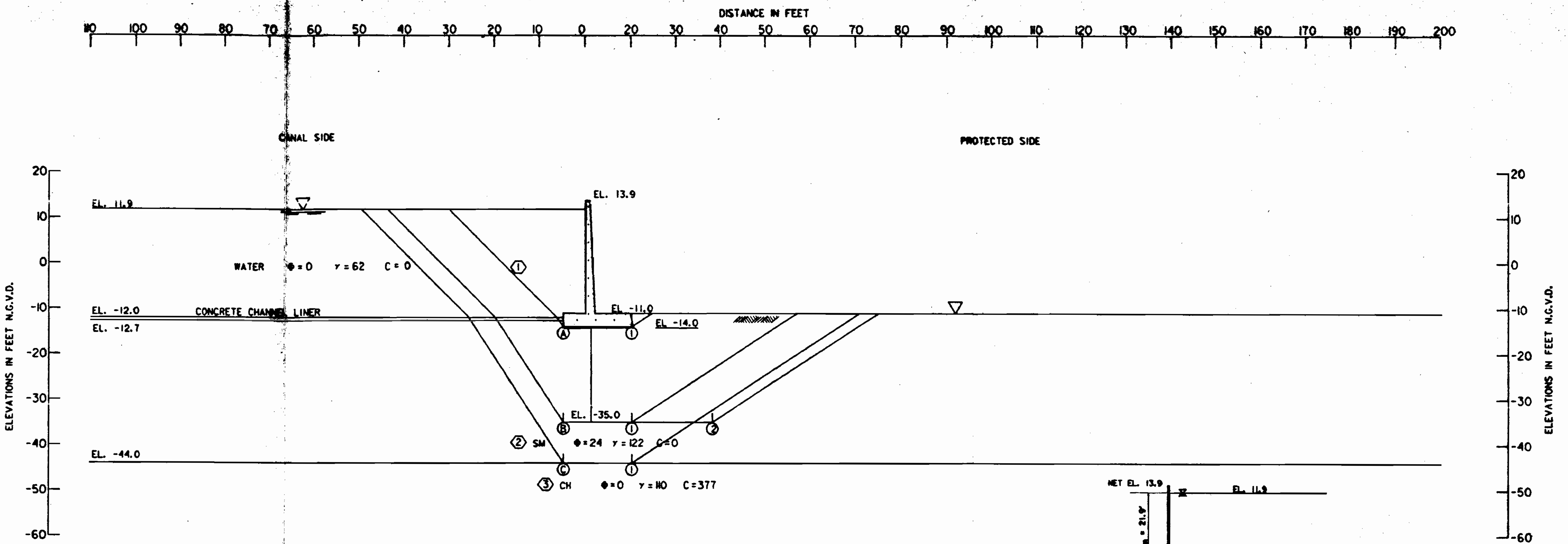
SEE PLATE 22 FOR SOIL DESIGN PARAMETERS.

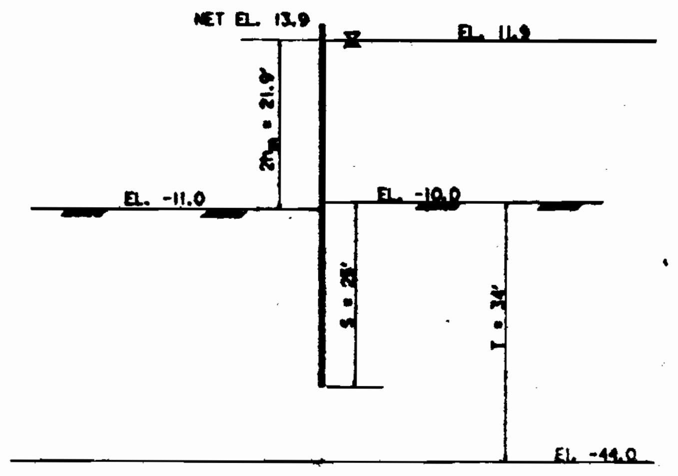
LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

ANCHORED WALL ANALYSIS
TEMPORARY COFFERDAM

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	PLOT SCALE: 10:1	PLOT DATE: DEC 94	CADD FILE: BRACE.DGN
DRAWN BY: WOODS			FILE NO. H-2-40511
CHECKED BY: RICHARDSON	DATE: DEC 94		





ASSUMED FAILURE SURFACE		$U_A = D_A - R_A$		$U_P = R_B + R_P + D_P$					
NO.	ELEV.	D_A	R_A	R_B	R_P	D_P	U_A	U_P	$U_A - U_P$
A 1	-14.0	18183	0	1287	367	549	18183	2203	15983
B 1	-35.0	79517	5529	9632	23495	35135	71988	68262	3726
B 2	-35.0	79517	7529	21185	23494	35134	71988	79813	-7825
C 1	-44.0	122383	15499	13208	44420	66427	106884	124055	-17171

GENERAL NOTES:

1. CLASSIFICATION, STRATIFICATION, SHEAR STRENGTH, AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS. SEE BORING DATA PLATES.
2. DEEP-SEATED STABILITY ANALYSIS UTILIZED A FACTOR OF SAFETY 1.3 INCORPORATED INTO THE SOIL PARAMETERS (C=COHESION AVAILABLE/1.3 AND φ = ARCTAN $\left(\frac{\tan\phi \text{ AVAILABLE}}{1.3}\right)$).

NOTES

○-- STRATUM NUMBER
⬡-- WEDGE NUMBER
∫-- CROSSOVER POINT
φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

SEEPAGE CALCULATED BASED ON HARR'S METHOD

S = 25' T = 34' $2h_m = 21.9'$

S/T = 25'/34' = .735

$\frac{i_e S}{h_m}$ = 0.54 for S/T = 0.735 FROM HARR'S CHART

$$i_e = \frac{0.54\ h_m}{S} = \frac{0.54\ (10.95')}{25'} = 0.24$$

i_{cr} = 59.5 PCF/62.5 PCF = 0.95

$$F.S. = \frac{i_{cr}}{i_e} = \frac{0.95}{0.24} = 4.0$$

4.0 = 4.0 FOR (SM) AND (SP) OK

LAKE PONTCHARTRAIN, LA AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

DEEP SEATED STABILITY ANALYSIS

STA. 0+92.05 SUB B/L TO STA. 9+47.39 SB B/L

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

FILE NO. H-2-40511

ENCL 4 REVISED MAY 95 PLATE 32

APPENDIX A

BORING LOGS & TEST RESULTS

STA 99+95 EB/L

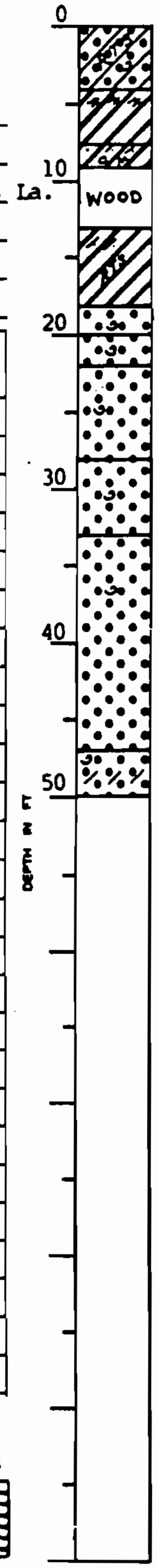
LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana
For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

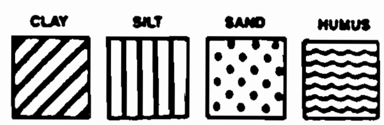
Boring No. 56 Soil Technician A. J. Mayeux Date 5 December 1985
Ground Elev. +4.6 Datum ______ Gr. Water Depth ______

Sample No.	Sample Depth — Feet From	Sample Depth — Feet To	Depth Stratum Feet From	Depth Stratum Feet To	Visual Classification	*Standard Penetration Test	
1	2.0	3.0	0.0	4.0	Medium stiff brown & gray clayey sand		
					w/roots & few shell fragments		
2	5.0	6.0	4.0	7.5	Soft brown & gray clay w/organic clay		
					layers		
3	8.0	9.0	7.5	9.0	Soft dark gray & brown clay w/sand		
					lenses, clayey sand pockets & trace		
					of organic matter		
	11.0	11.5	9.0	13.0	Wood w/clay & organic matter		
4	14.0	15.0	13.0	18.0	Extremely soft gray clay w/clayey sand		
					pockets & roots		
5	18.0	19.0	18.0	20.0	Very loose gray sand w/shell fragments		
6	20.0	21.5	20.0	22.0	Loose gray sand w/shell fragments	2	5
7	22.5	24.0	22.0		Medium dense gray sand w/shell	2	12
					fragments		
8	25.0	26.5		28.0	Ditto	5	18
9	28.5	30.0	28.0	33.0	Dense gray sand w/shell fragments	7	42
10	33.5	35.0	33.0		Very dense gray sand w/shell fragments	9	54
11	38.5	40.0			Ditto	8	56
12	43.5	45.0		47.0	Ditto	13	50=10"
13	48.5	50.0	47.0	50.0	Medium dense gray sand w/shell	2	11
					fragments & clay layers		



*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



Predominant type shown heavy. Modifying type shown light.

Remarks: 5" Diameter Boring

STA 103+57.90 EB/L

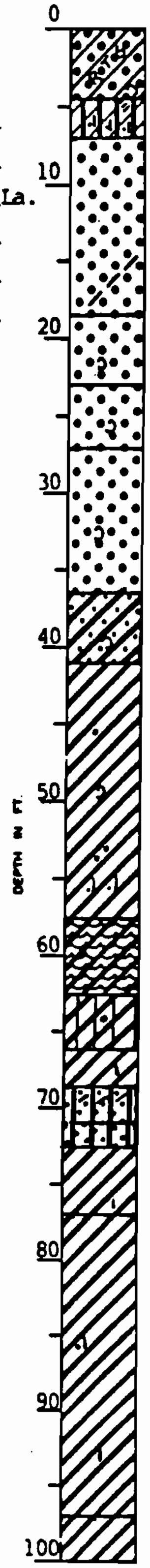
LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana
For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

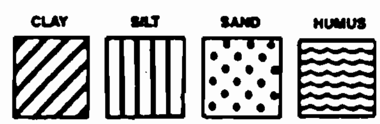
Boring No. 57 Soil Technician A. J. Mayeux Date 9 December 1985
Ground Elev. -4.0 Datum ______ Gr. Water Depth See Text

Sample No.	Sample Depth — Feet From	Sample Depth — Feet To	Depth Stratum Feet From	Depth Stratum Feet To	Visual Classification	*Standard Penetration Test	
1	2.0	2.5	0.0	4.5	Medium dense brown & gray clayey sand		
					w/roots & few shell fragments		
2	5.0	5.5	4.5	7.0	Medium compact gray clayey silt w/sandy		
					silt layers & clay pockets		
3	7.0	8.5	7.0		Very loose Gray sand w/clay layers	0	1
4	9.0	10.5			Ditto	0	1
5	11.5	13.0			Ditto	0	3
6	14.0	15.5		18.5	Ditto	1	2
7	18.5	20.0	18.5	23.0	Medium dense gray sand w/shell	2	11
					fragments		
8	23.5	25.0	23.0	27.0	Dense gray sand w/shell fragments	7	34
9	28.5	30.0	27.0		Medium dense gray sand w/shell	7	20
					fragments		
10	33.5	35.0		36.5	Ditto	7	12
11	38.5	40.0	36.5	41.0	Soft gray sandy clay w/shell fragments	0	2
12	44.0	44.5	41.0		Medium stiff gray clay w/trace of sand		
					& shell fragments		
13	49.0	49.5			Medium stiff gray clay w/sand pockets		
					& shell fragments		
14	54.0	54.5		57.5	Medium stiff gray clay w/fine sandy		
					silt pockets		
15	59.0	59.5	57.5	62.5	Stiff dark gray organic clay		
16	64.0	64.5	62.5	66.0	Stiff greenish-gray silty clay w/fine		
					sand		
17	67.0	67.5	66.0	68.5	Stiff greenish-gray & tan clay w/silt		
					lenses		



*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



Remarks: ______

Predominant type shown heavy. Modifying type shown light.

LOG OF BORING

EUSTIS ENGINEERING COMPANY
SOIL AND FOUNDATION CONSULTANTS
METAIRIE, LA.

Name of Project: London Avenue Canal, Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269, New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District, New Orleans, La.
Burk & Associates, Inc., New Orleans, Louisiana

Boring No. 57 (Cont'd) Soil Technician A. J. Mayeux Date 9 December 1985

Ground Elev. ______ Datum ______ Gr. Water Depth See Text

Sample No.	SAMPLE Depth — Feet From	To	DEPTH STRATUM Feet From	To	VISUAL CLASSIFICATION	*STANDARD PENETRATION TEST	
18	68.5	70.0	68.5	71.0	Loose greenish-gray sandy silt w/clay	2	7
					layers		
19	71.0	72.5	71.0	72.5	Medium compact greenish-gray sandy	4	12
					silt		
20	73.5	75.0	72.5	77.0	Medium stiff gray clay w/silt lenses	1	5
21	79.0	79.5	77.0		Stiff gray clay w/fine sandy silt		
					lenses		
22	84.0	84.5			Ditto		
23	89.0	89.5			Stiff gray clay w/silt lenses		
24	94.0	94.5		97.0	Ditto		
25	99.0	99.5	97.0	100.0	Medium stiff gray clay		

DEPTH IN FT

*Number in first column indicates number of blows of 140-lb. hammer dropped 30 in. required to seat 2-in. O. D. splitspoon sampler 6 in. Number in second column indicates number of blows of 140-lb. hammer dropped 30 in. required to drive 2-in. O. D. splitspoon sampler 1 ft. after seating 6 in.

WHILE THIS LOG OF BORING IS CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT ITS RESPECTIVE LOCATION ON THE DATE SHOWN, IT IS NOT WARRANTED THAT IT IS REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Remarks: ______

CLAY SILT SAND HUMUS

Predominant type shown heavy. Modifying type shown light.

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

BORING 56

EL +4.6

Sample No.	Depth in Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI
1	2.0	Medium stiff brown & gray clay w/clayey sand pockets	33.8	83.4	111.6	1690			
2	5.0	Soft brown & gray clay w/organic clay layers	94.1	46.1	89.5	540	124	34	90
3	8.0	Soft dark gray & brown clay w/sand lenses, clayey sand pockets & trace of organic matter	68.6	57.8	97.4	780			
4	14.0	Extremely soft gray clay w/clayey sand pockets & roots	63.4	62.8	102.5	155	79	24	55

Geotechnical Investigation
London Avenue Canal
Levee and Floodwall Improvements
Orleans Levee Board Project No. 2049-0269
New Orleans, Louisiana

For: The Board of Levee Commissioners of the Orleans Levee District
New Orleans, Louisiana

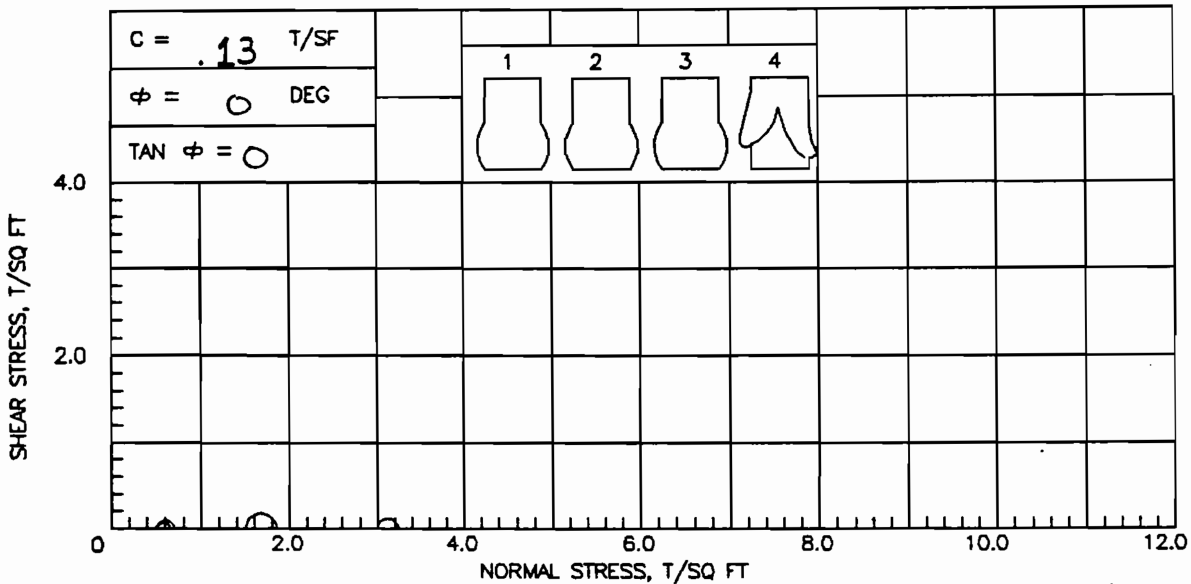
Burk & Associates, Inc., Engineers, Planners & Environmental Scientists
New Orleans, Louisiana

SUMMARY OF LABORATORY TEST RESULTS

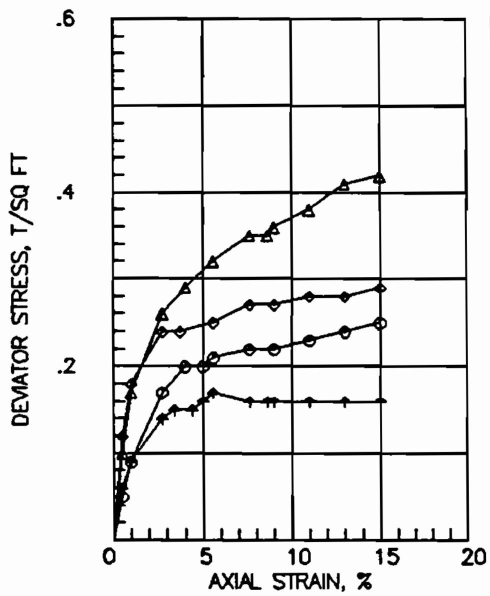
BORING 57
EL -4.0

Sample No.	Depth in Feet	Classification	Water Content Percent	Density PCF Dry	Density PCF Wet	Unconfined Compressive Strength PSF	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI
1	2.0	Medium dense brown & gray clayey sand w/roots & few shell fragments	18.9	----	-----	----			
2	5.0	Medium compact gray clayey silt w/sandy silt layers & clay pockets	27.9	92.5	118.3	1200*			
12	44.0	Medium stiff gray clay w/trace of sand & shell fragments	66.3	59.8	99.5	1760			
13	49.0	Medium stiff gray clay w/sand pockets & shell fragments	49.5	71.2	106.4	1080	71	21	50
14	54.0	Medium stiff gray clay w/fine sandy silt pockets	43.7	76.6	110.1	1260			
15	59.0	Stiff dark gray organic clay	101.2	43.3	87.1	3015	163	42	121
16	64.0	Stiff greenish-gray silty clay w/fine sand	19.1	109.1	129.9	3495			
17	67.0	Stiff greenish-gray & tan clay w/silt lenses	31.5	90.7	119.3	3590			
21	79.0	Stiff gray clay w/fine sandy silt lenses	37.7	83.5	115.0	2370*			
23	89.0	Stiff gray fissured clay w/silt lenses	44.7	77.0	111.5	3885	76	22	54
25	99.0	Medium stiff gray fissured clay w/silt lenses	45.4	75.5	109.8	1560			

*Unconsolidated Undrained Triaxial Compression Test - One Specimen; Confined at the approximate overburden pressure.



$\gamma = 113$



SPECIMEN NO.		○ 1	△ 2	◇ 3	+ 4
INITIAL	WATER CONTENT, %	37.3	32.6	41.2	45.5
	DRY DENSITY, PCF	83.5	89.3	79.0	74.6
	SATURATION, %	98.8	99.2	98.2	97.7
	VOID RATIO	1.020	.888	1.133	1.258
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	.50
MAX. DEV. STRESS, TSF		.20	.35	.24	.15
TIME TO FAILURE, MIN.		9	33	8	18
STRAIN RATE INCR., %					5
INITIAL DIAMETER, IN.		1.35	1.35	1.36	1.35
INITIAL HEIGHT, IN.		3.00	3.00	3.00	3.00

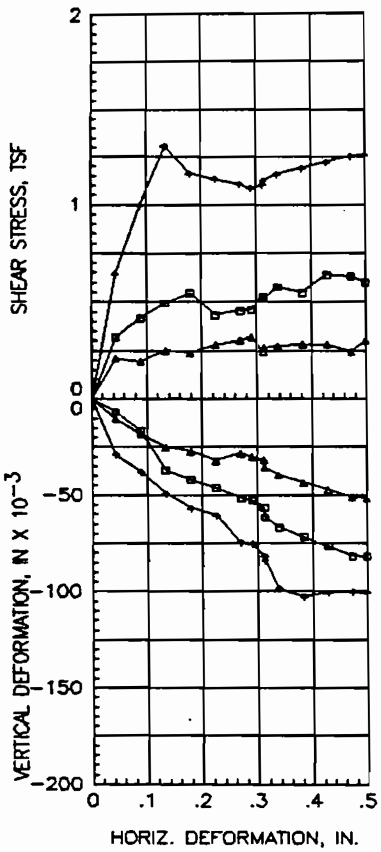
CONTROLLED-STRAIN TEST

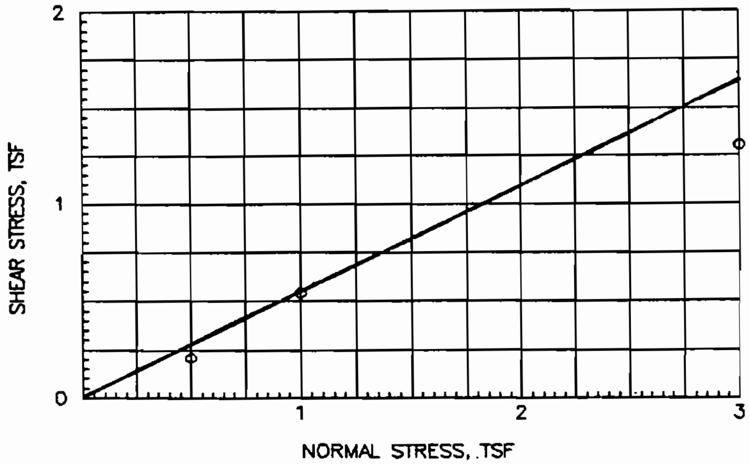
DESCRIPTION OF SPECIMENS: SANDY CLAY (CL), GRAY; ROOTLETS

LL 31	PL 19	PI 12	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:		
	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 2B
	DEPTH/ELEV 4.7/-8.4	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 10 MAR 94

TRIAXIAL COMPRESSION TEST REPORT





ϕ = 25.5 DEG

TAN ϕ = .477

C = 0 T/SF

	TEST NO.	1 ▲	2 □	3 ◆
INITIAL	WATER CONTENT, %	25.5	24.2	24.9
	VOID RATIO	.737	.768	.789
	SATURATION, %	92.6	84.0	84.2
	DRY DENSITY, PCF	95.9	94.2	93.1
VOID RATIO AFTER CONSOL				
FIFTY PERCENT CONSOL, MIN		< 1	< 1	< 1
FINAL	WATER CONTENT, %	29.3	30.0	30.2
	VOID RATIO			
	SATURATION, %			
NORMAL STRESS, TSF		.50	1.00	3.00
MAXIMUM SHEAR STRESS, TSF		.21	.55	1.31
TIME TO FAILURE, MIN		236	965	722
RATE OF STRAIN, IN/MIN		.00019	.00019	.00019
ULTIMATE SHEAR STRESS, TSF		.20	.43	1.09

TYPE SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	.552 IN. THICK
CLASSIFICATION	SILTY SAND (SM), GRAY; SHELL PARTICLES		

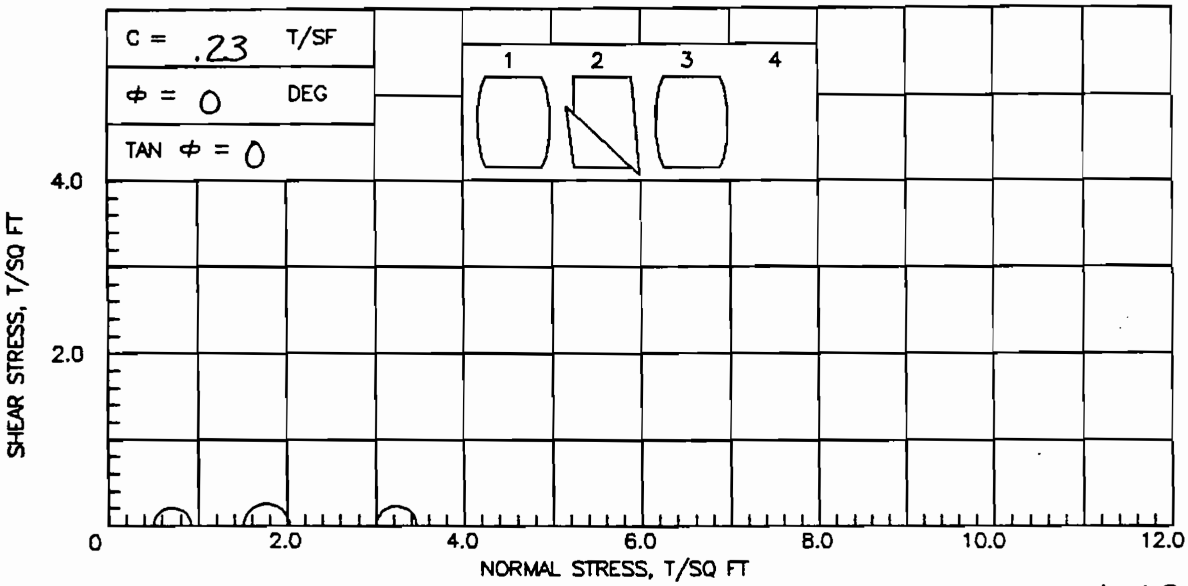
LL	PL	PI		GS 2.67 (EST)

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE 6B
	DEPTH/ELEV 20.5/-24.2	TECH. JL
	LABORATORY USAE WES - STF/GL	DATE 09 MAR 94
	DIRECT SHEAR TEST REPORT	

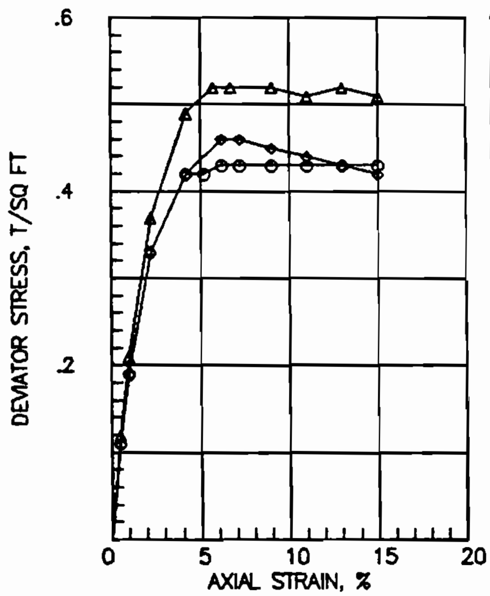
C = .23 T/SF

Φ = 0 DEG

TAN Φ = 0



γ= 112



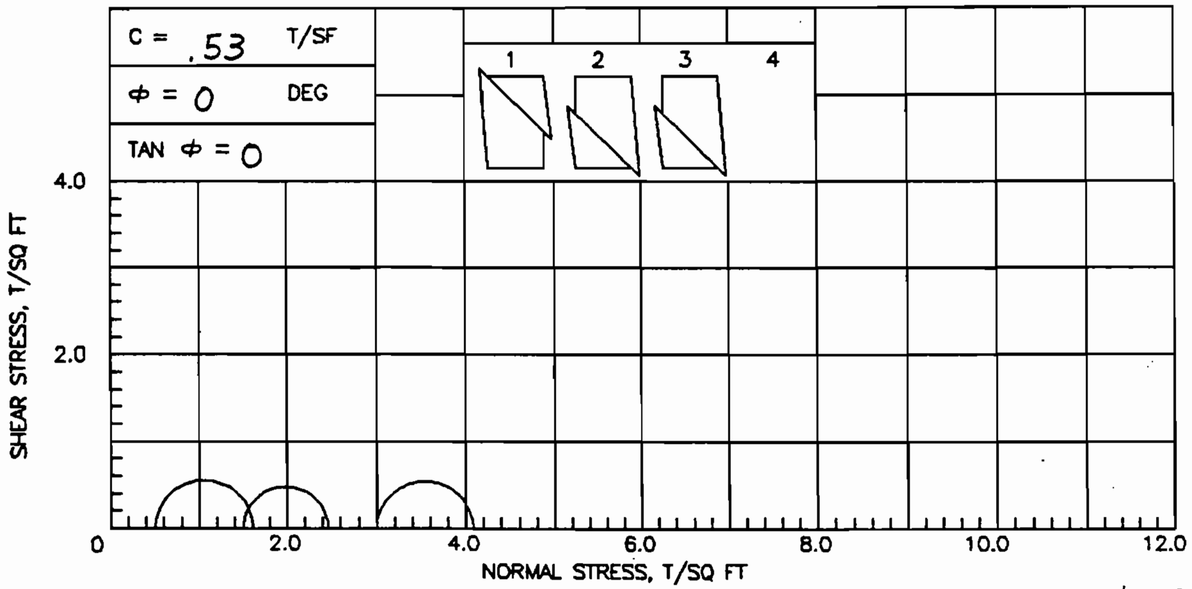
	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	41.4	39.3	40.9	
	DRY DENSITY, PCF	78.5	80.5	79.9	
	SATURATION, %	97.5	97.0	99.5	
	VOID RATIO	1.147	1.094	1.109	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
	CHAMBER PRESSURE, TSF	.50	1.50	3.00	
	MAX. DEV. STRESS, TSF	.42	.52	.46	
	TIME TO FAILURE, MIN.	9	19	21	
	STRAIN RATE INCR., %				
	INITIAL DIAMETER, IN.	1.36	1.36	1.35	
	INITIAL HEIGHT, IN.	3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

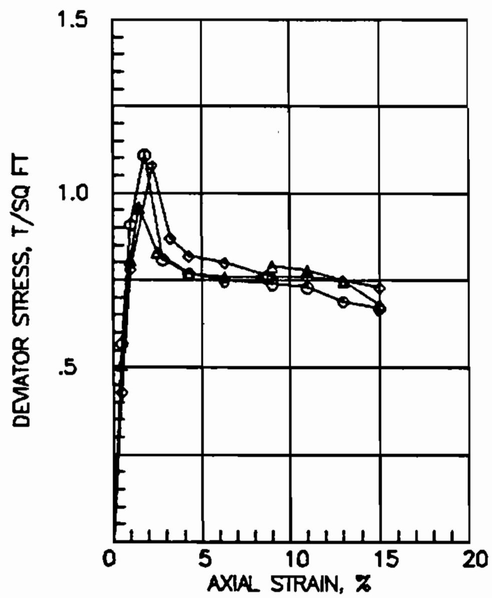
DESCRIPTION OF SPECIMENS: SANDY CLAY (CL), GRAY

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 11B
	DEPTH/ELEV 41.0/-44.7	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 11 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	



γ=102



		○ 1	△ 2	◇ 3	4
SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	65.2	59.3	60.1	
	DRY DENSITY, PCF	61.0	64.4	64.4	
	SATURATION, %	99.9	99.1	100+	
	VOID RATIO	1.762	1.617	1.619	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		1.11	.96	1.08	
TIME TO FAILURE, MIN.		4	8	15	
STRAIN RATE INCR., %			8	3	
INITIAL DIAMETER, IN.		1.36	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

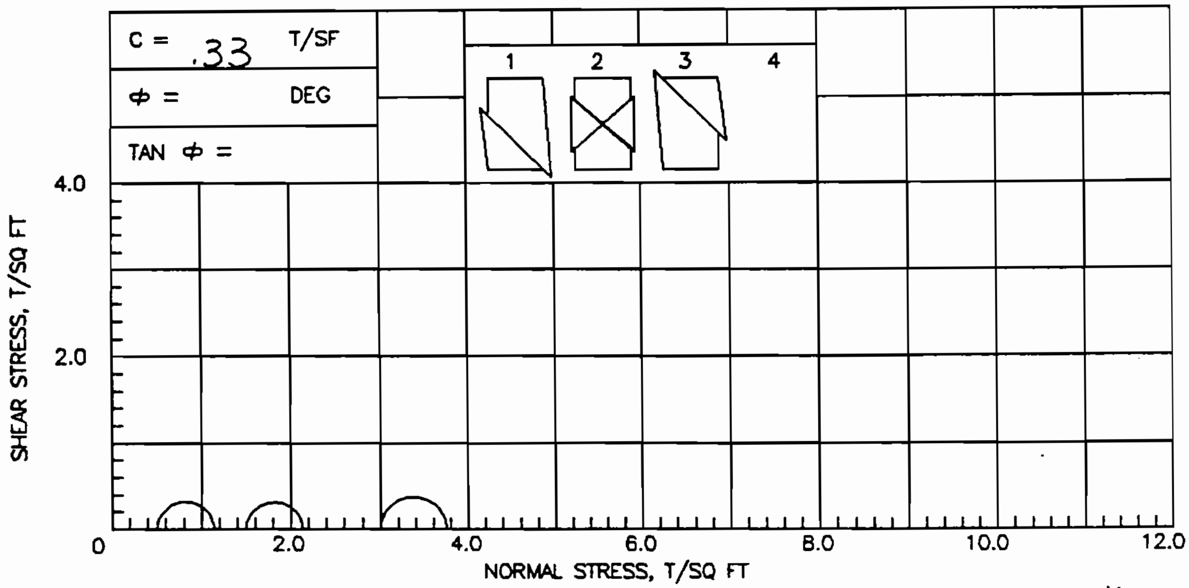
CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; SHELL PARTICLES

LL 73	PL 24	PI 49	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 12C
	DEPTH/ELEV 45.3/-49.0	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 14 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

C =	.33 T/SF
Φ =	DEG
TAN Φ =	



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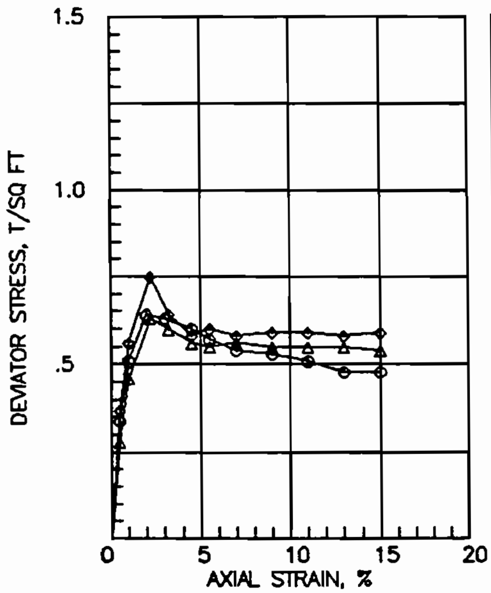


	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	54.5	56.5	54.6	
	DRY DENSITY, PCF	68.6	67.3	68.4	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.457	1.503	1.464	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
	CHAMBER PRESSURE, TSF	.50	1.50	3.00	
	MAX. DEV. STRESS, TSF	.64	.63	.75	
	TIME TO FAILURE, MIN.	4	11	15	
	STRAIN RATE INCR., %		5	4	
	INITIAL DIAMETER, IN.	1.36	1.36	1.36	
	INITIAL HEIGHT, IN.	3.00	3.00	3.00	

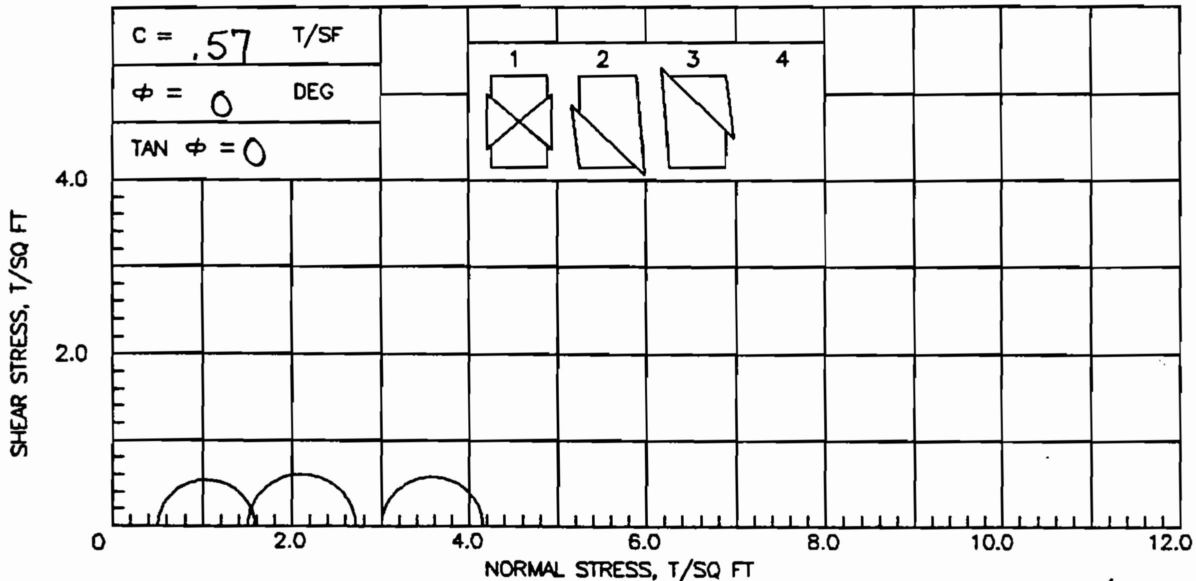
CONTROLLED-STRAIN TEST

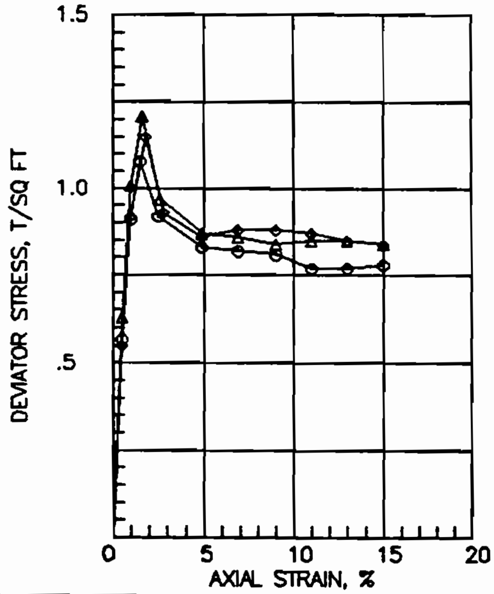
DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 13B
	DEPTH/ELEV 48.0/-51.7	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 15 MAR 94

TRIAXIAL COMPRESSION TEST REPORT





γ = 111

	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	44.2	42.2	43.6	
	DRY DENSITY, PCF	77.0	78.7	77.6	
	SATURATION, %	100+	99.9	100+	
	VOID RATIO	1.189	1.142	1.172	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
	CHAMBER PRESSURE, TSF	.50	1.50	3.00	
	MAX. DEV. STRESS, TSF	1.08	1.21	1.15	
	TIME TO FAILURE, MIN.	3	8	14	
	STRAIN RATE INCR., %		6	3	
	INITIAL DIAMETER, IN.	1.37	1.36	1.36	
	INITIAL HEIGHT, IN.	3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; SHELL PARTICLES

LL 58	PL 18	PI 40	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

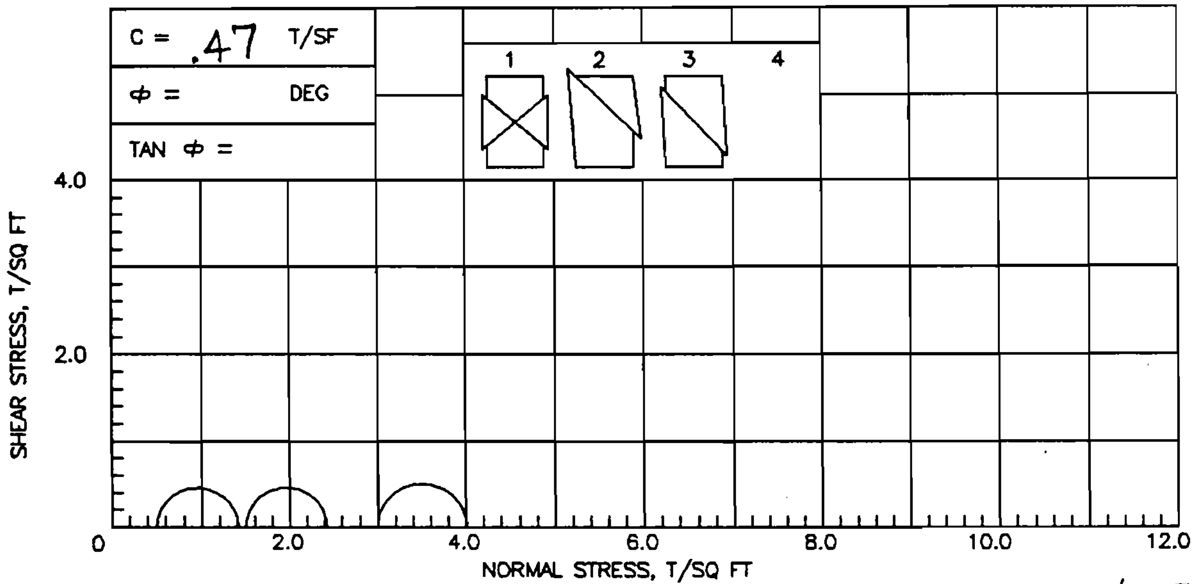
PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.	
BORING NO.	10-LUG	SAMPLE NO. 14B
DEPTH/ELEV	52.1/-55.8	TECH. JH
LABORATORY	USAE WES - STF/GL	DATE 16 MAR 94

TRIAXIAL COMPRESSION TEST REPORT

C = .47 T/SF

Φ = DEG

TAN Φ =



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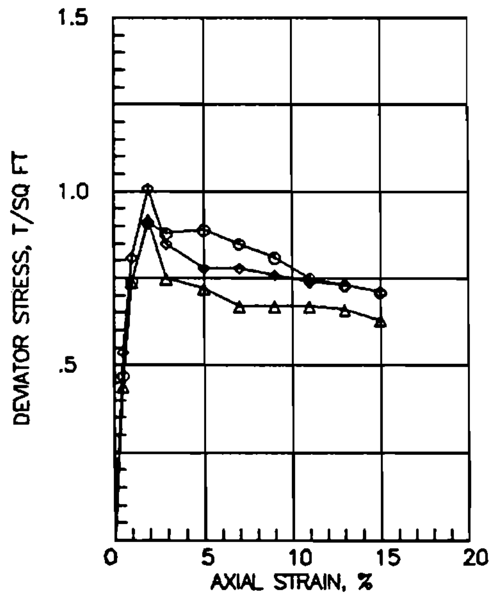


	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	56.3	57.6	56.5	
	DRY DENSITY, PCF	67.1	66.2	67.1	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.512	1.547	1.511	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
	CHAMBER PRESSURE, TSF	.50	1.50	3.00	
	MAX. DEV. STRESS, TSF	.91	.92	1.01	
	TIME TO FAILURE, MIN.	4	9	14	
	STRAIN RATE INCR., %			4	
	INITIAL DIAMETER, IN.	1.37	1.37	1.36	
	INITIAL HEIGHT, IN.	3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

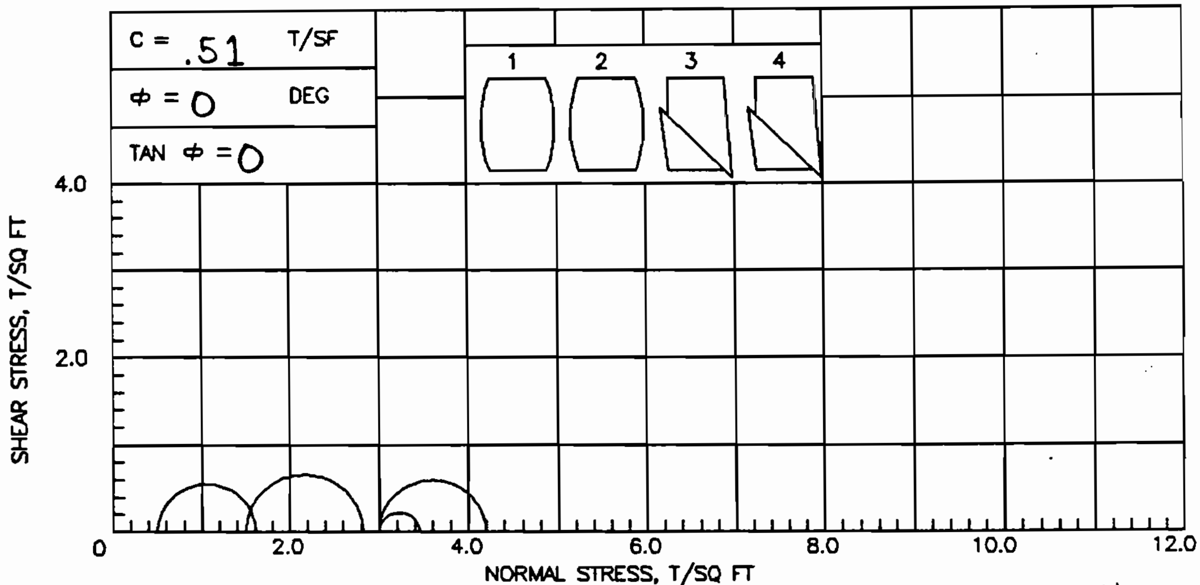
DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.
BORING NO. 10-LUG	SAMPLE NO. 15B
DEPTH/ELEV 56.0/-59.7	TECH. JH
LABORATORY USAE WES - STF/GL	DATE 17 MAR 94

TRIAXIAL COMPRESSION TEST REPORT



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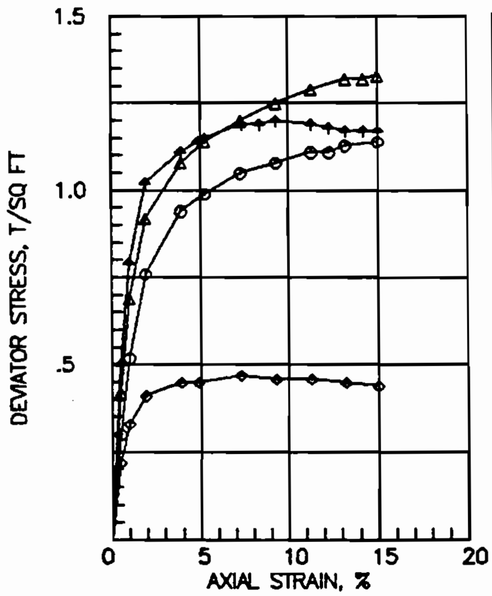


			⊙ 1	△ 2	◇ 3	+ 4
SPECIMEN NO.			⊙ 1	△ 2	◇ 3	+ 4
INITIAL	WATER CONTENT, %		25.6	23.9	29.7	24.9
	DRY DENSITY, PCF		97.5	99.8	91.0	98.9
	SATURATION, %		94.7	93.7	94.0	95.5
	VOID RATIO		.729	.689	.852	.704
BEFORE SHEAR	WATER CONTENT, %					
	DRY DENSITY, PCF					
	SATURATION, %					
	VOID RATIO					
	BACK PRESSURE, TSF					
CHAMBER PRESSURE, TSF			.50	1.50	3.00	3.00
MAX. DEV. STRESS, TSF			1.11	1.32	.45	1.19
TIME TO FAILURE, MIN.			25	29	9	23
STRAIN RATE INCR., %						
INITIAL DIAMETER, IN.			1.36	1.36	1.36	1.36
INITIAL HEIGHT, IN.			3.00	3.00	3.00	3.00

CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: SILTY CLAY (CL), GRAY

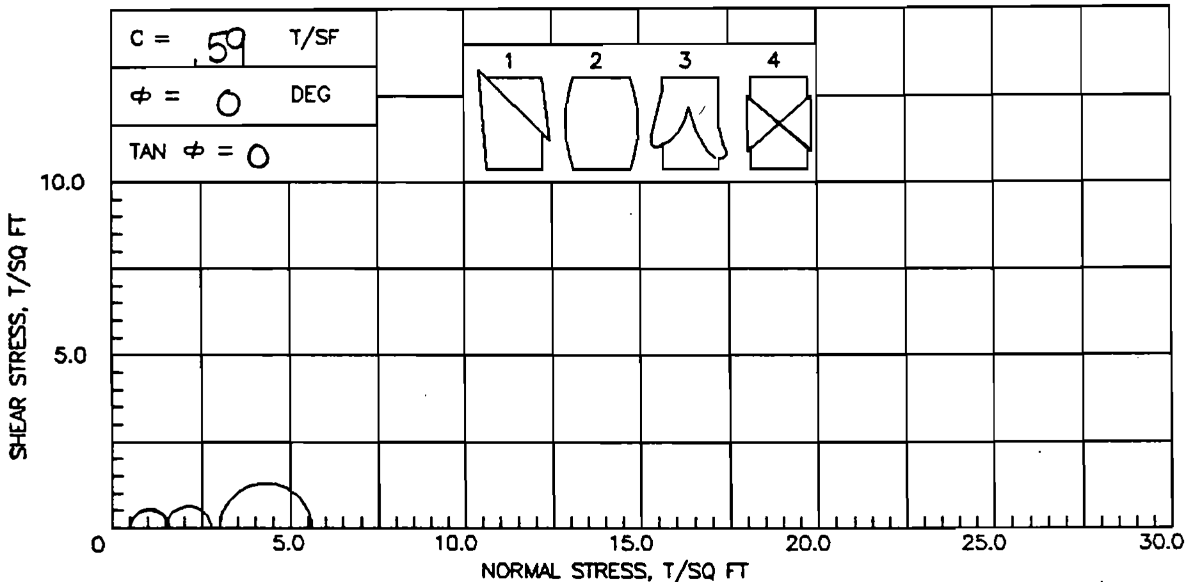
LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10–LUG	SAMPLE NO. 16C
	DEPTH/ELEV 61.0/–64.7	TECH. JH
	LABORATORY USAE WES – STF/GL	DATE 18 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

C = .59 T/SF

ϕ = 0 DEG

TAN ϕ = 0



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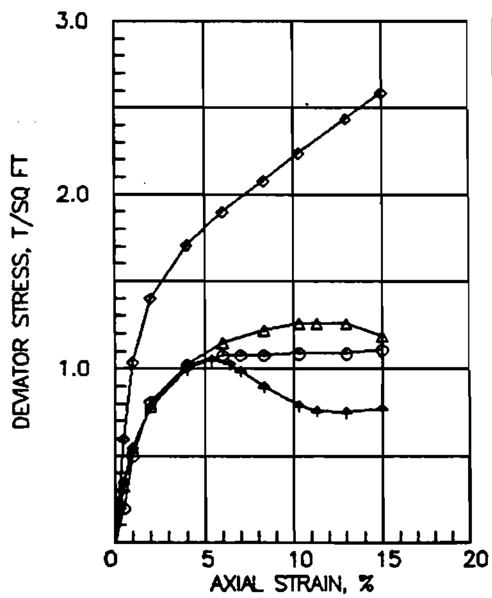


SPECIMEN NO.		○ 1	△ 2	◇ 3	+ 4
INITIAL	WATER CONTENT, %	29.6	32.9	23.4	27.6
	DRY DENSITY, PCF	89.8	87.4	98.8	93.6
	SATURATION, %	91.3	95.7	89.8	93.2
	VOID RATIO	.877	.928	.705	.801
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	.50
MAX. DEV. STRESS, TSF		1.08	1.26	2.59	1.05
TIME TO FAILURE, MIN.		13	23	33	17
STRAIN RATE INCR., %					
INITIAL DIAMETER, IN.		1.37	1.37	1.37	1.37
INITIAL HEIGHT, IN.		3.00	3.00	3.00	3.00

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), GRAY; SILT POCKETS & SEAMS

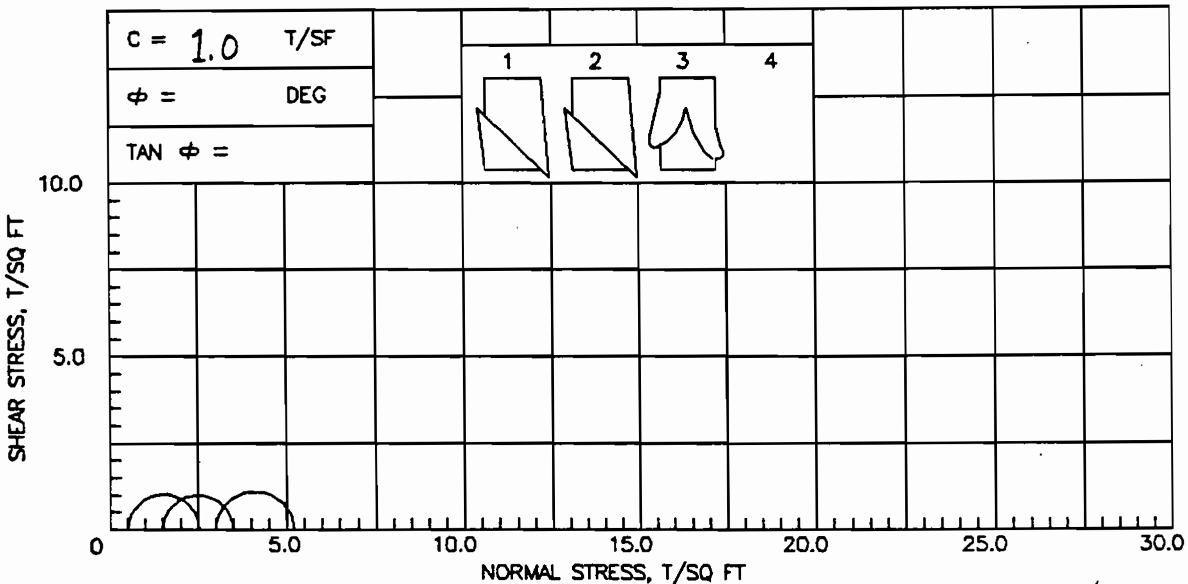
LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 17D
	DEPTH/ELEV 65.9/-69.6	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 21 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

c = 1.0 T/SF

φ = DEG

TAN φ =



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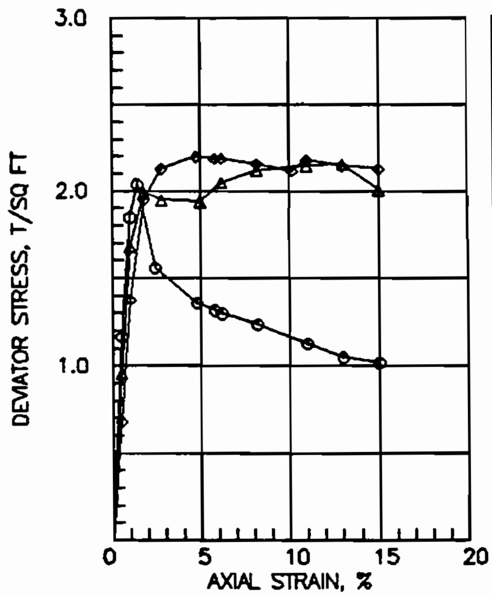


SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	32.2	32.7	32.6	
	DRY DENSITY, PCF	89.6	88.7	88.3	
	SATURATION, %	98.6	98.0	97.0	
	VOID RATIO	.881	.901	.908	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		2.04	1.99	2.20	
TIME TO FAILURE, MIN.		3	9	38	
STRAIN RATE INCR., %			5	10	
INITIAL DIAMETER, IN.		1.37	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; SILT SEAMS; SLICKENSIDED

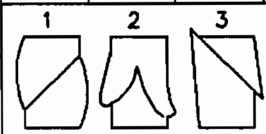
LL 67	PL 21	PI 46	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

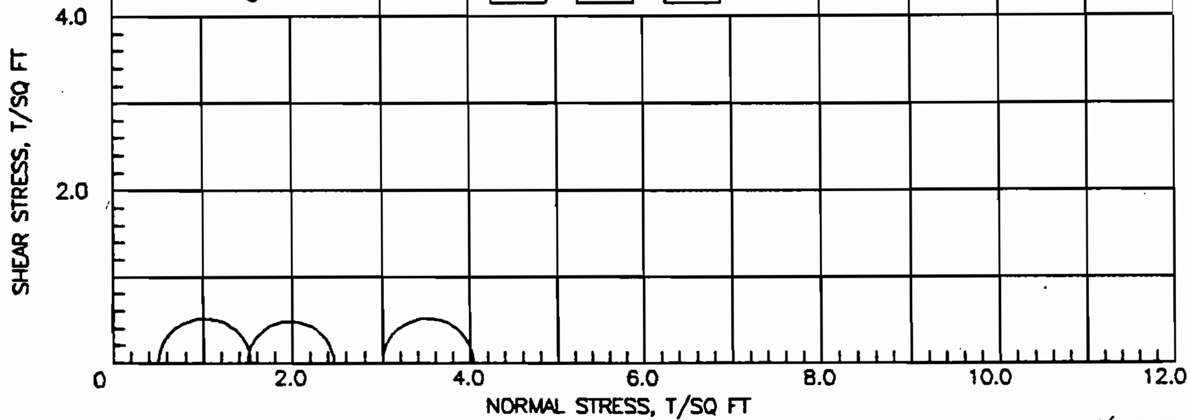
REMARKS:

PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.
BORING NO. 10-LUG	SAMPLE NO. 18B
DEPTH/ELEV 68.5/-72.2	TECH. JH
LABORATORY USAE WES - STF/GL	DATE 23 MAR 94

TRIAXIAL COMPRESSION TEST REPORT

C = .5	T/SF
Φ = 0	DEG
TAN Φ = 0	





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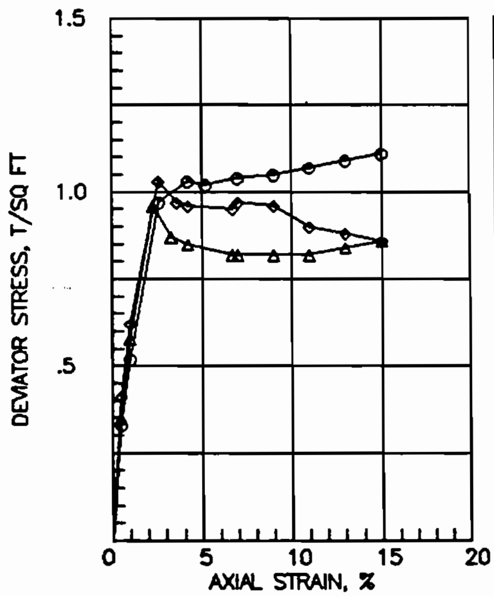


SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	35.7	38.1	35.9	
	DRY DENSITY, PCF	84.5	83.3	85.2	
	SATURATION, %	97.0	100+	99.1	
	VOID RATIO	.995	1.024	.978	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		1.03	.96	1.03	
TIME TO FAILURE, MIN.		9	12	13	
STRAIN RATE INCR., %				6	
INITIAL DIAMETER, IN.		1.36	1.36	1.37	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

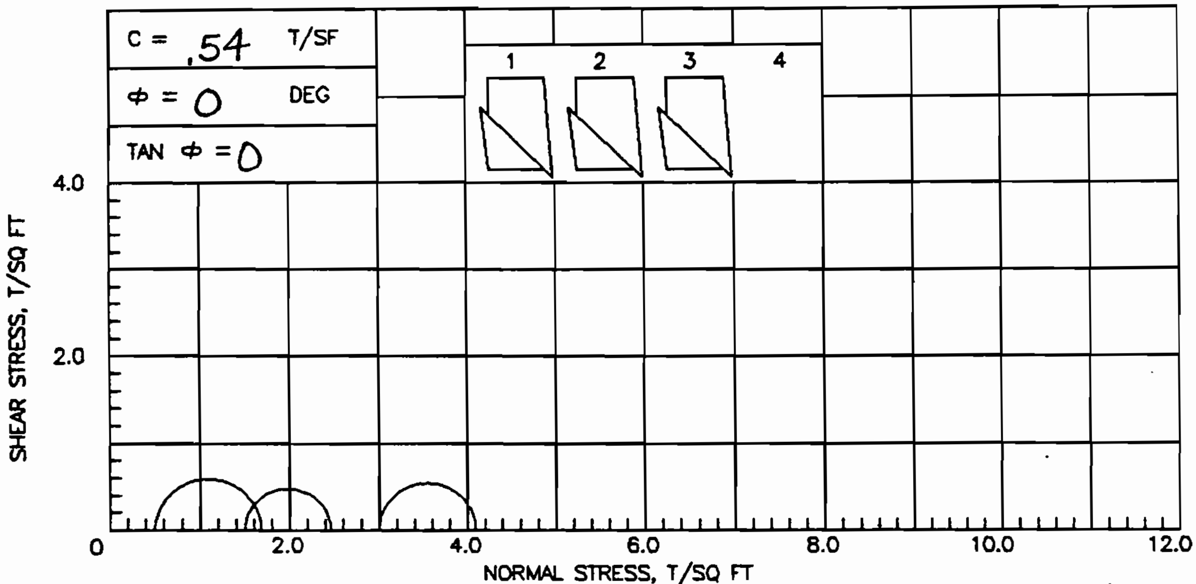
CONTROLLED–STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CL), GRAY; SILT SEAMS & POCKETS

LL 48	PL 19	PI 29	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10–LUG	SAMPLE NO. 20B
	DEPTH/ELEV 76.0/–79.7	TECH. JH
	LABORATORY USAE WES – STF/GL	DATE 24 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

C = .54	T/SF
Φ = 0	DEG
TAN Φ = 0	



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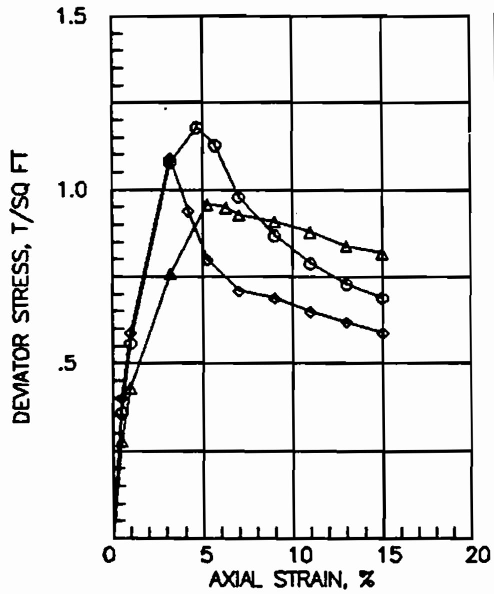


SPECIMEN NO.		○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	36.1	39.6	40.5	
	DRY DENSITY, PCF	83.4	80.6	79.4	
	SATURATION, %	95.5	98.0	97.4	
	VOID RATIO	1.020	1.091	1.123	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		1.18	.96	1.09	
TIME TO FAILURE, MIN.		10	18	10	
STRAIN RATE INCR., %					
INITIAL DIAMETER, IN.		1.38	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; 3/4" SILT LAYER

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.	
BORING NO. 10-LUG	SAMPLE NO. 21B
DEPTH/ELEV 80.0/-83.7	TECH. JH
LABORATORY USAE WES - STF/GL	DATE 25 MAR 94

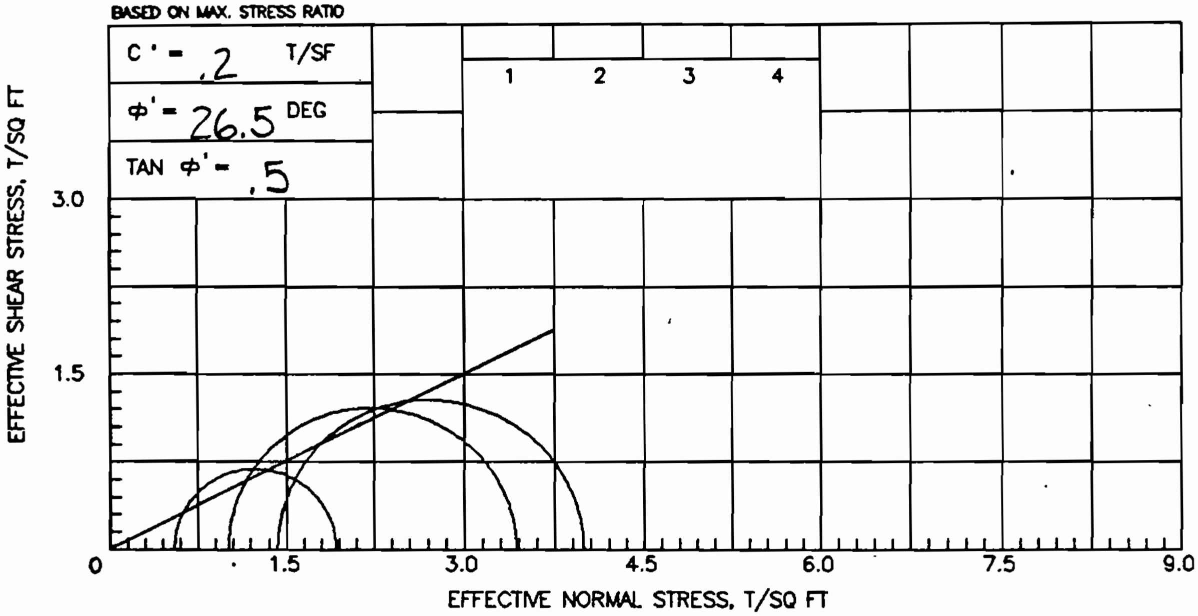
TRIAXIAL COMPRESSION TEST REPORT

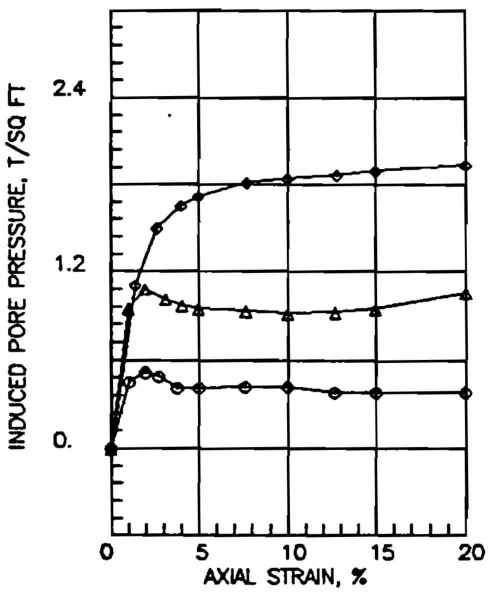
BASED ON MAX. STRESS RATIO

C' = .2 T/SF

φ' = 26.5 DEG

TAN φ' = .5





	SPECIMEN NO.	⊙ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	46.0	40.3	37.9	
	DRY DENSITY, PCF	74.5	77.9	80.6	
	SATURATION, %	98.4	93.6	93.9	
	VOID RATIO	1.261	1.163	1.091	
BEFORE SHEAR	WATER CONTENT, %	44.1	37.9	35.2	
	DRY DENSITY, PCF	77.4	81.6	86.4	
	SATURATION, %	100+	96.1	100+	
	VOID RATIO	1.179	1.065	.950	
	BACK PRESS., TSF	7.20	7.20	7.20	
@ σ'₁/σ'₃ MAX	MIN PRIN EFF STRESS, TSF	.55	1.01	1.42	
	DEV. STRESS, TSF	1.38	2.42	2.56	
	TIME TO FAILURE, MIN.	173	126	323	
RATE OF STRAIN, %/MIN		.016	.016	.016	
INITIAL DIAMETER, IN.		1.38	1.39	1.39	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

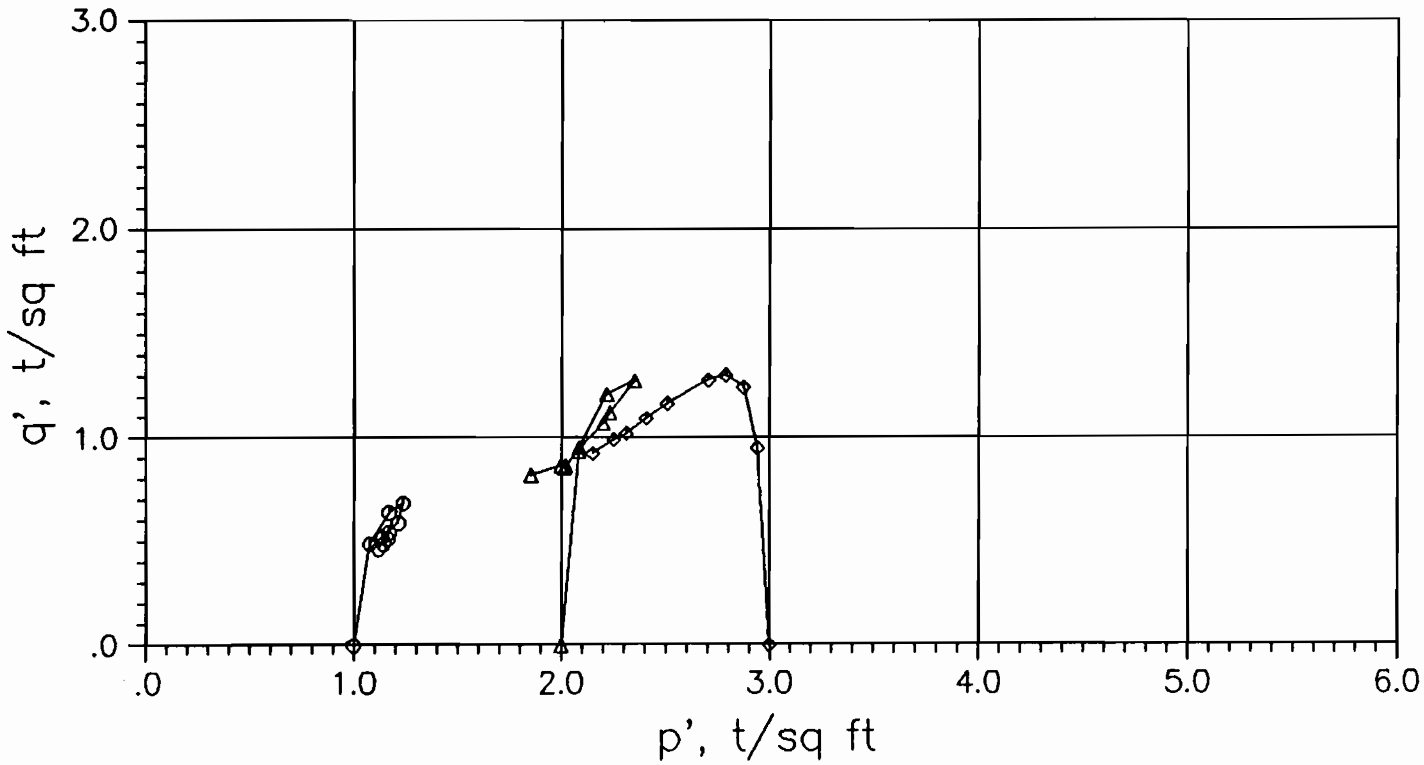
DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; SILT LENSES

LL 51	PL 18	PI 33	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	R̄ TEST

REMARKS:

PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.	
BORING NO.	10-LUG	SAMPLE NO. 23C
DEPTH/ELEV	89.4/-93.1	TECH. JL
LABORATORY	USAE WES - STF/GL	DATE 14 MAR 94

R̄-TRIAXIAL COMPRESSION TEST REPORT

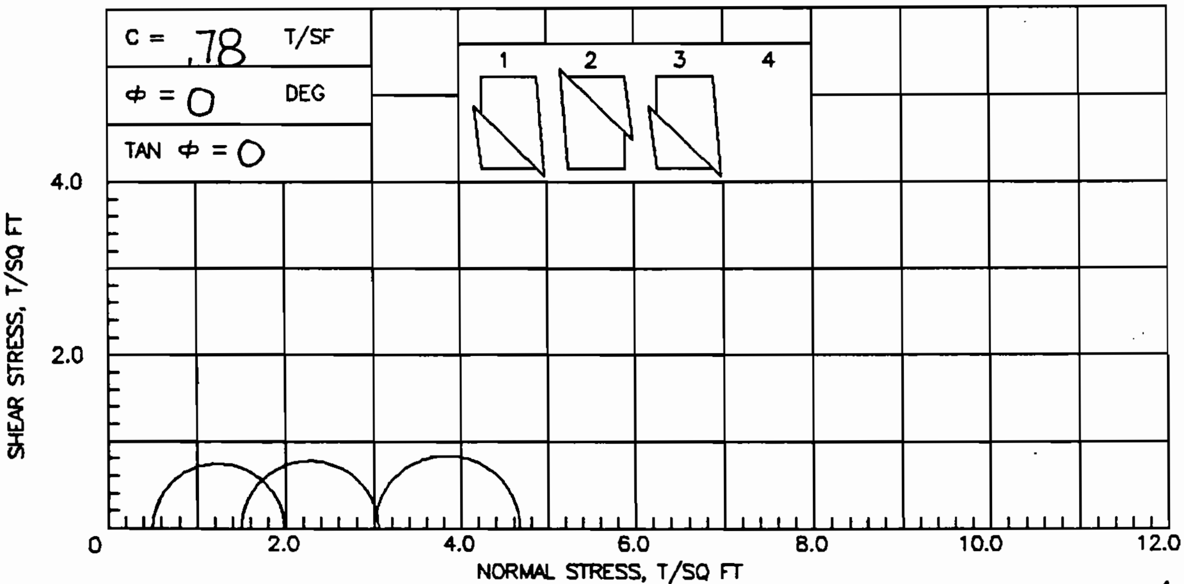


PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL		
	PUMP STA. #4 AT PRENTISS AVE.		
BORING	10-LUG	SAMPLE NO.	23C
DEPTH/ELEV	89.4/-93.1	DATE	14 MAR 94

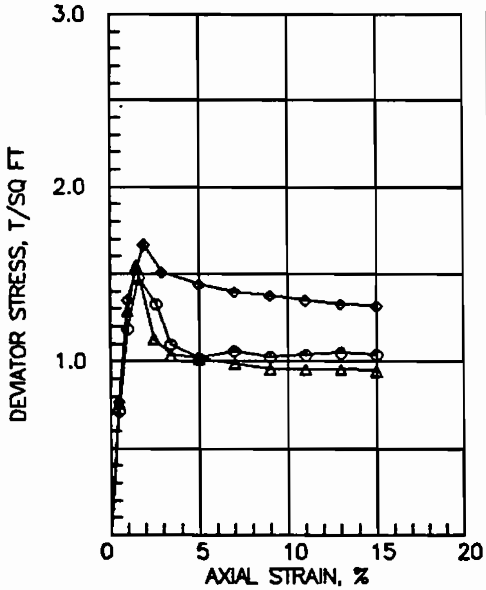
Stress Paths

LABORATORY USAE WES – STF/GL

C = .78	T/SF	
Φ = 0	DEG	
TAN Φ = 0		



γ=114



	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	40.0	40.2	40.4	
	DRY DENSITY, PCF	81.3	80.8	81.3	
	SATURATION, %	100+	99.9	100+	
	VOID RATIO	1.073	1.086	1.074	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		1.48	1.55	1.67	
TIME TO FAILURE, MIN.		4	8	15	
STRAIN RATE INCR., %				3	
INITIAL DIAMETER, IN.		1.36	1.37	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY; SILT SEAMS

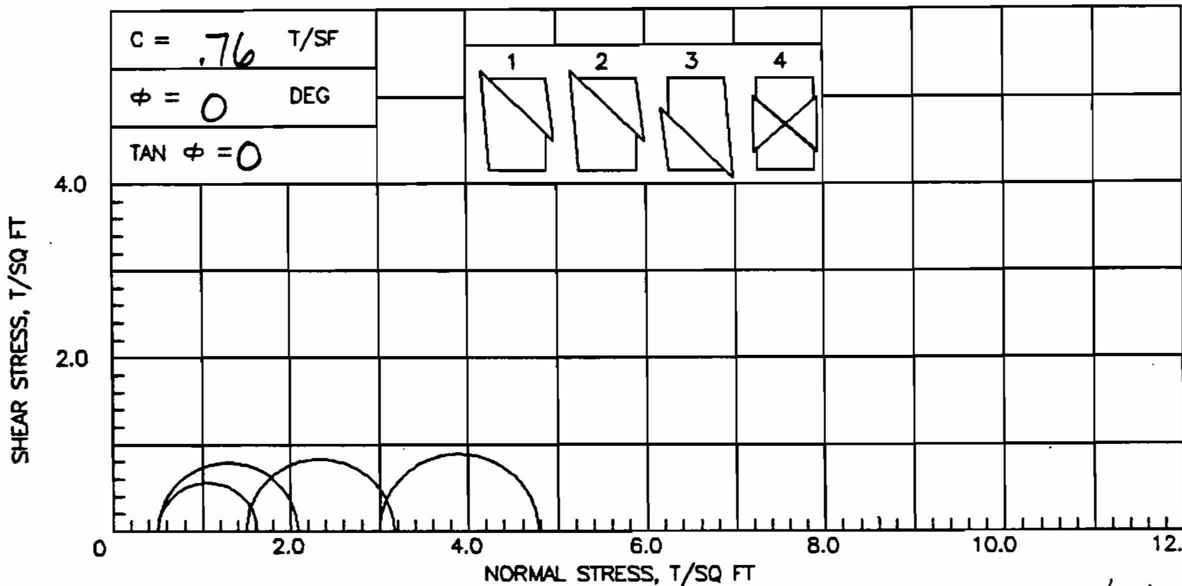
LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 26C
	DEPTH/ELEV 100.8/-104.5	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 28 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

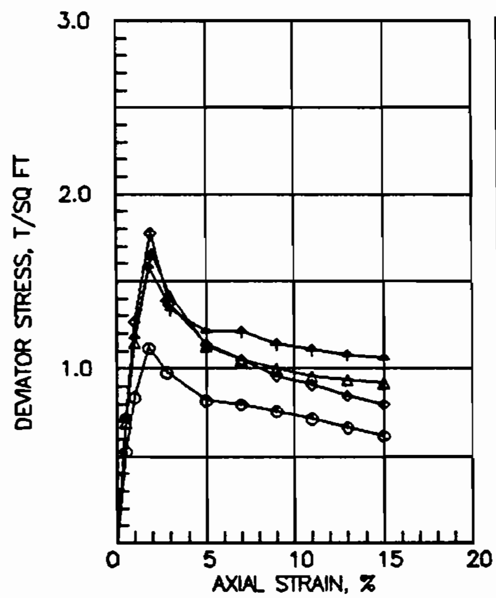
C = .76 T/SF

Φ = 0 DEG

TAN Φ = 0



γ=113



		SPECIMEN NO.	⊙ 1	△ 2	◇ 3	+ 4
INITIAL		WATER CONTENT, %	42.0	41.7	42.8	41.5
		DRY DENSITY, PCF	79.3	79.9	78.9	80.2
		SATURATION, %	100+	100+	100+	100+
		VOID RATIO	1.125	1.110	1.136	1.102
BEFORE SHEAR		WATER CONTENT, %				
		DRY DENSITY, PCF				
		SATURATION, %				
		VOID RATIO				
		BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF			.50	1.50	3.00	.50
MAX. DEV. STRESS, TSF			1.12	1.66	1.78	1.58
TIME TO FAILURE, MIN.			4	11	14	13
STRAIN RATE INCR., %					3	5
INITIAL DIAMETER, IN.			1.36	1.36	1.36	1.37
INITIAL HEIGHT, IN.			3.00	3.00	3.00	3.00

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY

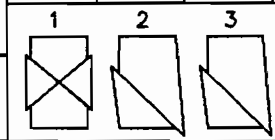
LL 62	PL 23	PI 39	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

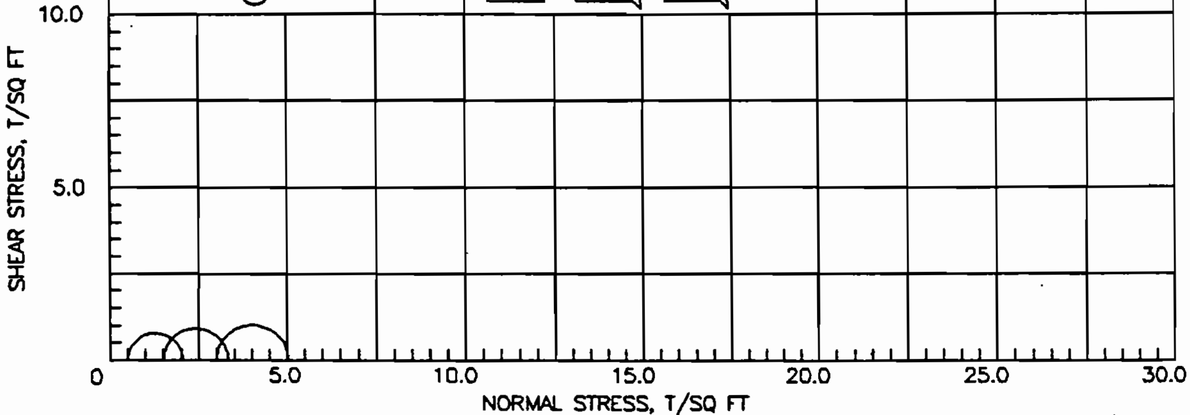
REMARKS:	PROJECT LAKE PONT. LONDON AVE OUTFALL CANAL	
	PUMP STA. #4 AT PRENTISS AVE.	
	BORING NO. 10-LUG	SAMPLE NO. 27C
	DEPTH/ELEV 105/-108.7	TECH. JH
	LABORATORY USAE WES - STF/GL	DATE 29 MAR 94
	TRIAXIAL COMPRESSION TEST REPORT	

C = .9 T/SF

φ = 0 DEG

TAN φ = 0





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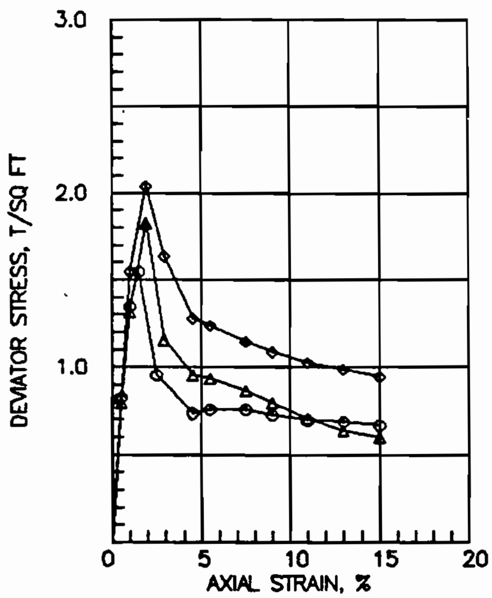


	SPECIMEN NO.	○ 1	△ 2	◇ 3	4
INITIAL	WATER CONTENT, %	46.8	46.1	46.9	
	DRY DENSITY, PCF	75.0	75.7	75.3	
	SATURATION, %	100+	100+	100+	
	VOID RATIO	1.247	1.228	1.237	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESSURE, TSF				
CHAMBER PRESSURE, TSF		.50	1.50	3.00	
MAX. DEV. STRESS, TSF		1.55	1.83	2.04	
TIME TO FAILURE, MIN.		3	15	14	
STRAIN RATE INCR., %			7	4	
INITIAL DIAMETER, IN.		1.37	1.36	1.36	
INITIAL HEIGHT, IN.		3.00	3.00	3.00	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: CLAY (CH), GRAY

LL	PL	PI	GS 2.70 (ESTIMATED)	UNDISTURBED SPECIMEN	Q TEST

REMARKS:

PROJECT	LAKE PONT. LONDON AVE OUTFALL CANAL PUMP STA. #4 AT PRENTISS AVE.
BORING NO. 10-LUG	SAMPLE NO. 28C
DEPTH/ELEV 109/-112.7	TECH. JH
LABORATORY USAE WES - STF/GL	DATE 30 MAR 94

TRIAXIAL COMPRESSION TEST REPORT

APPENDIX B

STRUCTURAL DESIGN CALCULATIONS

STRUCTURAL DESIGN CALCULATIONS:

1. T-WALL

2. GATED MONOLITH FOR 1000 CFS PUMPS

3. GATED DISCHARGE BASIN

29 Aug 94

T-WALL MONOLITH DESIGN

1) GENERAL:

A) DESIGN AS 2 COMPONENTS: STEM, SLAB

STEM TRANSFERS LOAD VERTICALLY TO SLAB.
ASSUME STEM FIXED @ BASE.

SLAB TRANSFERS LOAD HORIZONTALLY TO PILES.
ASSUME SLAB PINNED TO PILES. USE PROGRAM
"CFRAME" TO ANALYZE STRIPS @ CRITICAL LOCATIONS.

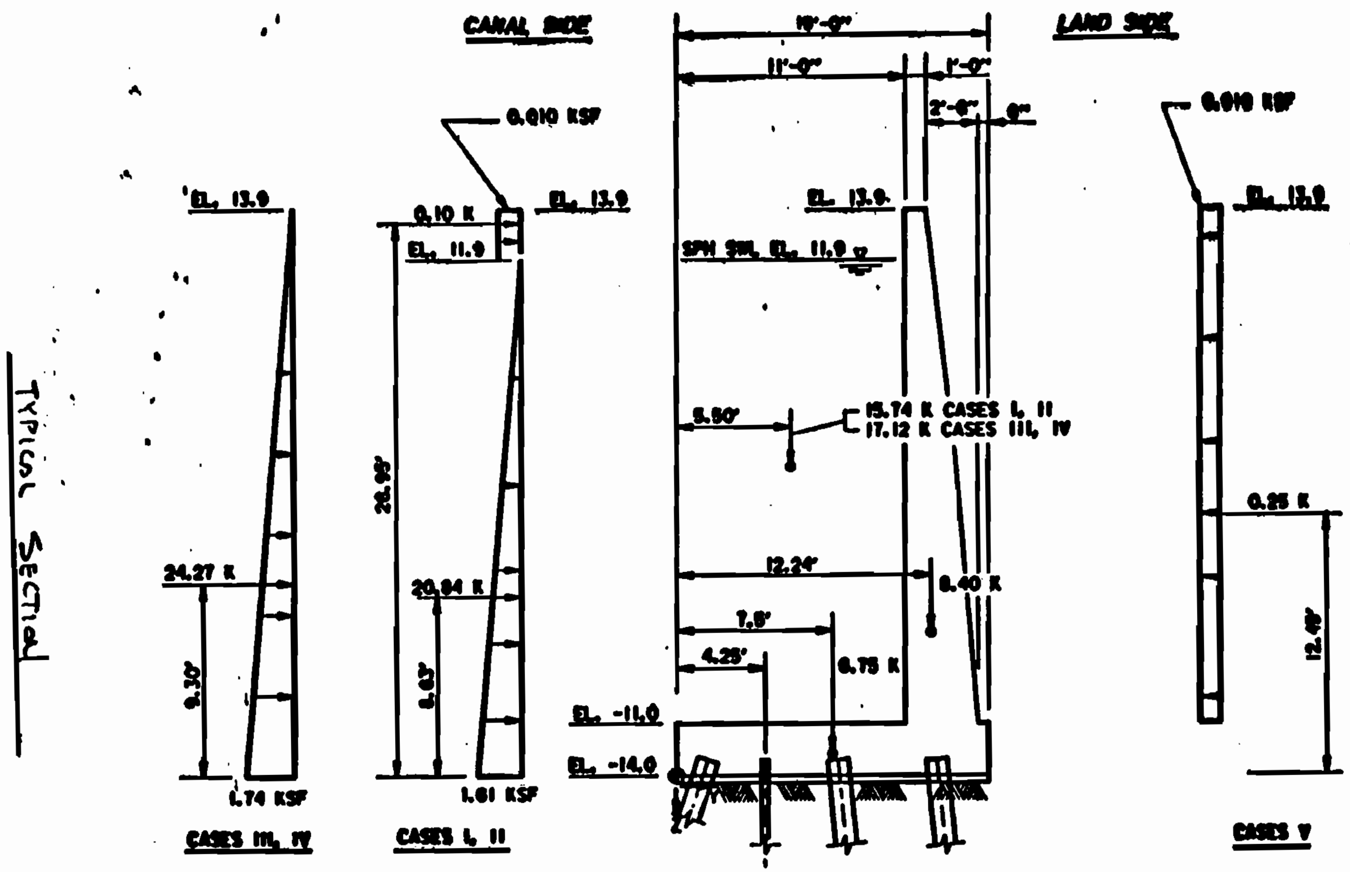
B) STEM AND SLAB DESIGNS WILL BE THE SAME
FOR BOTH T-WALL MONOLITHS

2) PARAMETERS:

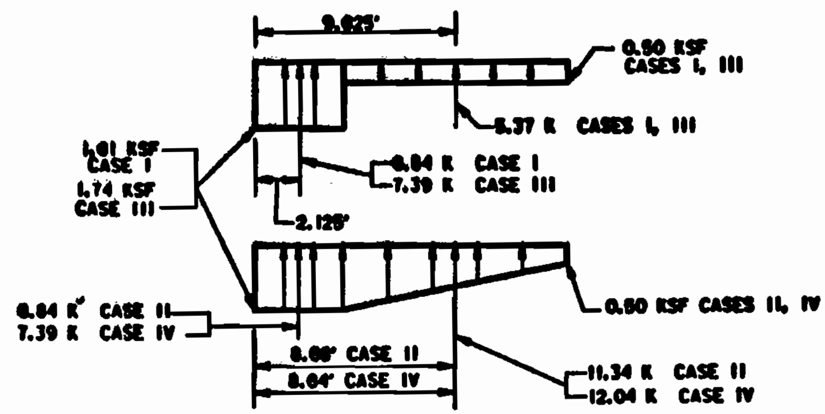
A. $f'_c = 3$ ksi
B. $f_y = 60.$ ksi
C. $\rho = .25\,\rho_b$
D. SINGLY REINFORCED SECTIONS

3) SPLICE LENGTHS, ANCHORAGE LENGTHS, ETC. TO BE
SPECIFIED IN CONSTRUCTION PLANS PREPARATION.

CANAL SIDE



1'- 0"



NOTE:
LOADS ARE PER FT. OF WALL LENGTH

FT. (PERPENDICULAR TO WALL)				
y (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	Mz (kips-ft)
.9	18.7	355.0	0.0	0.0
.9	12.7	309.0	0.0	0.0
.7	19.5	404.0	0.0	0.0
.7	12.9	352.0	0.0	0.0
25	15.2	151.0	0.0	0.0

NOTES:
1. PILE ANALYSIS RESULTS FROM COMPUTER PROGRAM CPGA IN THE CORPS LIBRARY

B	C	D

]1[Wed Sep 14 14:04:44 CDT 1994]

I) STEM : PER FT STRIP

1. FLEXURE : LOAD CASE 1

A. @ BASE SLAB, EL. -11.0

1. $M_u = 1.3(1.7)(1.43)(22.9)^2/6 = 276$ K.FT

2. CAPACITY BASE ON $\rho = .25\rho_b$

$$\rho_b = \frac{.85\beta_1 f'_c\, 87}{f_y(87+f_y)} = \frac{.85^2(3)(87)}{60(87+60)} = .0214$$

$A_s = .25(.0214)(12)(31.5) = 2.02$ IN^2

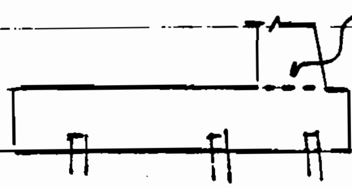
$a = 2.02(60)/.85(3)(12) = 3.96$ IN

$\phi M_n = .9(2.02)(60)[31.5 - 3.96/2] = 3220$ K.IN

$= 268.$ K.FT

3. $\phi M_n \approx M_u$ SECTION IS ADEQUATE

B. SHEAR :



SHEAR PATH FOR BOTH FLEXURAL SHEAR AND SHEAR FRICTION (POTENTIAL COLD JOINT LOCATION)

BY INSPECTION, ANY SHEAR FAILURE AT THE BASE STEM WILL BE CONTROLLED BY SHEAR-FRICTION, NOT FLEXURAL SHEAR; I.E. IF A CRACK ATTEMPTS TO PROPAGATE, SHEAR-FRICTIONAL FORCES MUST BE EXCEEDED.

1. ASSUME A COLD-JOINT IS USED AT STEM BASE:

$$V_n = A_{vf} f_y \mu$$

A_{vf} = USE TEMP. STEEL ON TENSION FACE

$= .5(.0028)(12)(36) = .604 \text{ in}^2$

$\mu \Rightarrow$ INTENTIONALLY ROUGHED SURFACE $= 1.\lambda$

$\lambda = 1.0$ NORMAL WT CONCRETE

$\therefore \phi V_n = .85(.60)(60)(1) = 30.6$ k

2. $V_u = 1.3(1.7)(.0625)(19.8)^2/2 = 27.1$ k

3. $\phi V_n > V_u$ OK

* 4. INSURE THAT AT LEAST $.604 \text{ in}^2$ OF TENSION FACE STEEL IS USED <u>IN ADDITION</u> TO THAT REQUIRED BY FLEXURE.

3) SLAB DESIGN

A) TRANSVERSE DIRECTION:

ANALYZE A 1' STRIP OF SLAB FOR CASE 1.

.. THE SHEAR AND MOMENT DIAGRAMS WILL BE OF SIMILAR SHAPE FOR LOAD CASES 1-4. I.E, THE LOCATIONS OF MAX. VALUES WILL NOT CHANGE SIGNIFICANTLY.

CFRAME MODEL OF T-WALL SLAB :

TRANSVERSE - 1 FT STRIP ; LOAD CASE 1

1. USE 3'0" THICK BASE SLAB w/ MEMBER PROPERTIES PROPORTIONAL TO $b = 12$, $d = 36 - 4.5 = 31.5$

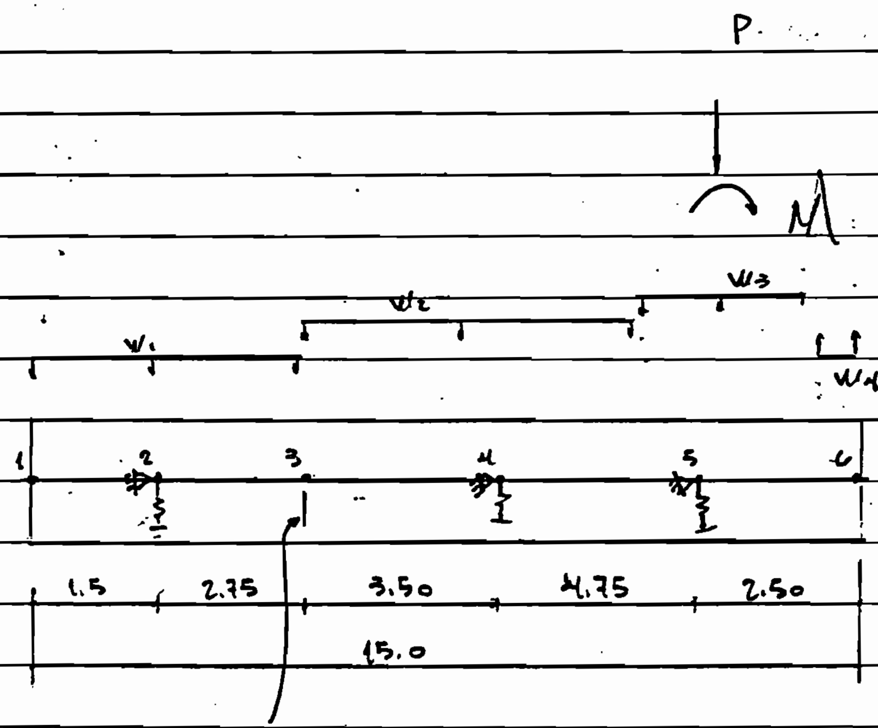
$$I = bd^3/12 = 12(31.5)^3/12 = 31255 \text{ in}^4$$
$$A = AS = 12(31.5) = 378 \text{ in}^2$$

THESE MEMBER PROPERTIES ARE INTENDED TO REPRESENT A CRACKED SECTION, NOT FULLY LOADED, I.E. UNTIL A SECTION IS FULLY LOADED THE ACTUAL SECTIONAL PROPERTIES ARE BETWEEN GROSS AREA PROPERTIES AND CRACKED, TRANSFORMED PROPERTIES BASED ON FULL SERVICE LOAD.

2. USE VERTICAL PILE SPRINGS AS FOLLOWS:

FROM CPGA : $B33 = CAE/L_E$, $C = 2.0$ (FRICTION)
$A = 21.4 \text{ in}^2$, $L_E = 90 \text{ FT} = 1080. \text{ in}$
$B33 = 2(21.4)(29000)/1080 = 1150 \text{ k/in}$

3. USE PINNED HORIZONTAL SUPPORTS => HORZ. MOVEMENT WILL NOT SIGNIFICANTLY AFFECT SHEAR AND MOMENT DIAGRAMS => THE STEM SHEAR LOAD IS IGNORED



LOCATION OF S.P. CUT-OFF

$P = 8.40 \text{ k}$ WT. OF STEM

$M = 20.84(8.6 - 3) = 116.7 \text{ k}\cdot\text{FT}$

$$w_1 = \overbrace{(11.9 + 11)(.0625)}^{1.431} + \overbrace{3(.15)}^{.45} - \overbrace{(11.9 + 14)(.0625)}^{1.61} = 0.27 \text{ k/FT}$$

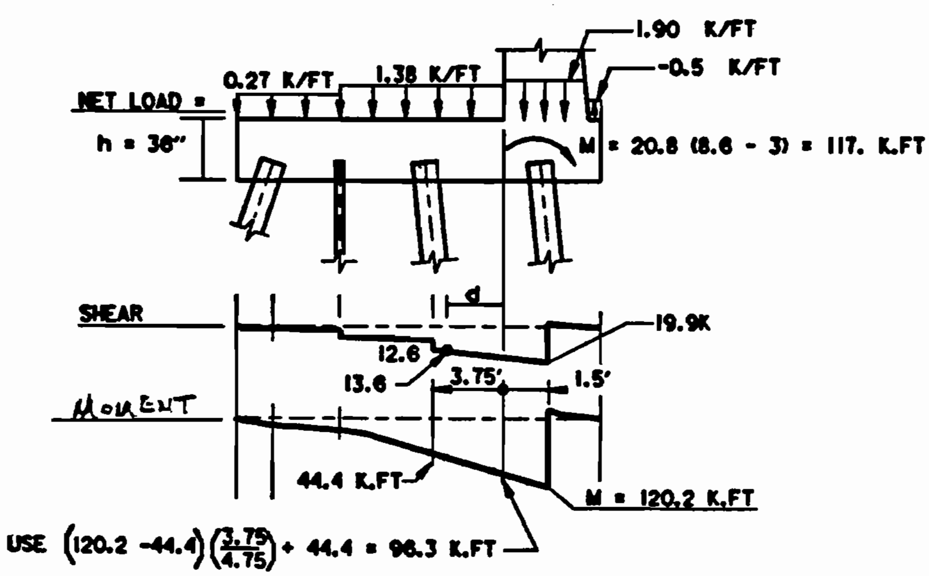
$$w_2 = 1.431 + .45 - \overbrace{(14. - 6)(.0625)}^{0.50} = 1.38 \text{ k/FT}$$

BOTTOM OF SLAB   GROUNDWATER EL.

$w_3 = 8.40 / 3.0 - 0.5 = 2.3 \text{ k/FT}$

$w_4 = -0.5 \text{ k/FT}$

TRANSVERSE DIRECTION - CASE I - ONE (1) FT STRIP



FLEXURE :

1. CAPACITY BASED ON $\rho = .25\,\rho b$

$\phi M_n = 268.$ k. FT P. 2

2. $M_u = 1.3(1.7)(96.3) = 213$ k. FT

3. $\phi M_n > M_u$ SECTION IS ADEQUATE

SHEAR: FLEXURAL CRACKING

1. $\phi V_c = 2\sqrt{f'_c}\, b d = 2(3000)^{1/2}(12)(31.5) = 35196$ LB

2. $V_u = 1.3(1.7)(13.6) = 30.1$ k

3. $\phi V_c > V_u$ NO SHEAR REINFORCEMENT

COMMENTS:

1. $M_u / \phi M_n (\rho = .25\rho_b) = 213/268 = 0.80$
@ JT. 5, AND LESS EVERYWHERE ELSE.
∴ CALCULATING THE EFFECTIVE MOMENT OF INERTIA, I_e, BASED ON MOMENT ALONG THE BEAM, TO DETERMINE NEW MOMENT DIAGRAM AND RE-EVALUATE SECTION IS UNNECESSARY.

2. COMPARE SUPPORT REACTIONS TO CPGA RESULTS:

FOR T-1, USE PILES 4, 15, AND 24 BECAUSE OF THEIR CENTRAL LOCATION.

PILE 24 = 112 k CPGA — MAX. COMPRESSION
JOINT 5 = 23.4 (5.75) = 134.5 k

PILE 15 = 87 k CPGA
JOINT 15 = -1.9 (5.75) = -11.0 k

PILE 4 = - 78.3 CPGA

JOINT 2 = - 4.7 (5.75) = - 27. k

ΣV : CPGA = 120. k

CFRAME = 96.5 k

24.5 k

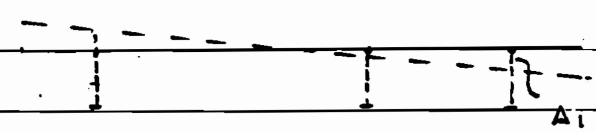
ASSUME ADDITIONAL IS DISTRIBUTED TO JOINTS 5 AND 15 EVENLY. => 24.5 / 2 = 12.2 k COMPRESSION

JOINT 5 = 134.5 + 12.2 = 146.7 k

JOINT 15 = -11 + 12.2 = 1.2 k

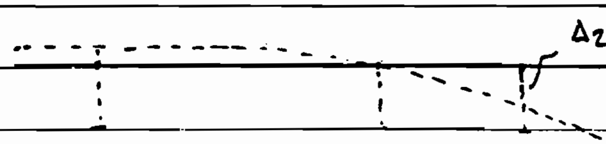
THE RIGID PILE CAP ASSUMPTION OF CPGA MAKES A SIGNIFICANT DIFFERENCE IN PILE LOAD DISTRIBUTION WHEN COMPARED TO CFRAME RESULTS.

DEFORMATIONS:



CPGA

$\Delta_1 < \Delta_2$



CFRAME

FOR BASE SLAB DESIGN, THE MOMENTS DEVELOPED IN THE CFRAME MODEL ARE APPROPRIATE.

FOR FOUNDATION DESIGN, ASSUMING CFRAME RESULTS ARE MORE REALISTIC, FOR PILE 24 (I.E. JOINT 5) ALF => 146.7 / 132. = 1.11.
AT THE POINT OF OVERLOAD, 146.7 - 132 = 14.7 k WILL GO TO PILE 15 (I.E. JOINT 15), TOTAL LOAD => 1.2 + 14.7 = 15.9 ; ALF = 15.9 / 132. = 0.12

END RESULT :

PILE 24 ALF 1.0 OK

PILE 15 ALF 0.12

```
010 T-WALL SLAB DESIGN - 2 SPAN BEAM
020 KSI FT IN IN KIP
030 6 5 1 3000 0.15
040 1 0. 0., 2 1.5  0., 3 4.25 0., 4 7.75 0., 5 12.5 0., 6 15. 0.
050 FIX X 2 4 5
060 FIX KY 1150 2 4 5
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6
130 31255 378 378 1 2 3 4 5
150 LOAD CASE 1 0 7 0 1 0  NET LOADS
160 0.0    0.27  1.50   0.27  0.  1
165 0.0    0.27  2.75   0.27  0.  2
170 0.0    1.38  3.50   1.38  0.  3
180 0.0    1.38  3.25   1.38  0.  4
185 3.25   1.90  4.75   1.90  0.  4
190 0.0    1.90  2.00   1.90  0.  5
200 2.00  -0.50  2.50  -0.50  0.  5
210 0.0   0.0  -117.    5
```

```
1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
 PROGRAM CFRAME V02.05 24JUL84
 *-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
```

RUN DATE = 94/08/08
RUN TIME = 8.35.37

T-WALL SLAB DESIGN - 2 SPAN BEAM

*** JOINT DATA ***

JOINT	X	Y	X	Y	R	KX	KY	KR
	--- FT ---		---FIXITY---			---KIP / IN---		IN-KIP/RAD
1	.00	.00						
2	1.50	.00	*				.115E+04	
3	4.25	.00						
4	7.75	.00	*				.115E+04	
5	12.50	.00	*				.115E+04	
6	15.00	.00						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	1.50	.3126E+05	.3780E+03	.3780E+03	.3000E+04	.1304E+04
2	2	3	2.75	.3126E+05	.3780E+03	.3780E+03	.3000E+04	.1304E+04
3	3	4	3.50	.3126E+05	.3780E+03	.3780E+03	.3000E+04	.1304E+04
4	4	5	4.75	.3126E+05	.3780E+03	.3780E+03	.3000E+04	.1304E+04
5	5	6	2.50	.3126E+05	.3780E+03	.3780E+03	.3000E+04	.1304E+04

*** LOAD CASE 1 NET LOADS

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.2700E+00	1.50	.2700E+00	.00
2	.00	.2700E+00	2.75	.2700E+00	.00
3	.00	.1380E+01	3.50	.1380E+01	.00
4	.00	.1380E+01	3.25	.1380E+01	.00
4	3.25	.1900E+01	4.75	.1900E+01	.00
5	.00	.1900E+01	2.00	.1900E+01	.00
5	2.00	-.5000E+00	2.50	-.5000E+00	.00

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT FT-KIP

5	.0000E+00	.0000E+00	-.1170E+03

LOAD CASE 1 NET LOADS

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	.0000E+00	.3891E-02	.1147E-04
2	.0000E+00	.4104E-02	.1123E-04
3	.0000E+00	.4482E-02	-.2125E-04
4	.0000E+00	.1623E-02	-.1745E-03
5	.0000E+00	-.2039E-01	-.7544E-03
6	.0000E+00	-.4317E-01	-.7573E-03

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
	2	.0000E+00	.4050E+00	-.3645E+01	-.3645E+01	18.00
2	2	.0000E+00	-.5125E+01	-.3645E+01	-.3645E+01	.00
	3	.0000E+00	.5867E+01	-.1850E+03	-.1850E+03	33.00
3	3	.0000E+00	-.5867E+01	-.1850E+03	-.1850E+03	.00
	4	.0000E+00	.1070E+02	-.5329E+03	-.5329E+03	42.00
4	4	.0000E+00	-.1256E+02	-.5329E+03	-.5329E+03	.00
	5	.0000E+00	.1990E+02	-.1443E+04	-.1443E+04	57.00
5	5	.0000E+00	.3550E+01	-.3885E+02	.9435E+00	22.20
	6	.0000E+00	.0000E+00	.0000E+00	-.3885E+02	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.0000E+00	-.4720E+01	.0000E+00
4	.0000E+00	-.1867E+01	.0000E+00
5	.0000E+00	.2345E+02	.0000E+00
TOTAL	.0000E+00	.1686E+02	

1 MEMBER END FORCES

MEMBER	LOAD CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN

1	1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
		2	.0000E+00	.4050E+00	-.3645E+01	-.3645E+01	18.00
2	1	2	.0000E+00	-.5125E+01	-.3645E+01	-.3645E+01	.00
		3	.0000E+00	.5867E+01	-.1850E+03	-.1850E+03	33.00
3	1	3	.0000E+00	-.5867E+01	-.1850E+03	-.1850E+03	.00
		4	.0000E+00	.1070E+02	-.5329E+03	-.5329E+03	42.00
4	1	4	.0000E+00	-.1256E+02	-.5329E+03	-.5329E+03	.00
		5	.0000E+00	.1990E+02	-.1443E+04	-.1443E+04	57.00
5	1	5	.0000E+00	.3550E+01	-.3885E+02	.9435E+00	22.20
		6	.0000E+00	.0000E+00	.0000E+00	-.3885E+02	.00

B) LONGITUDINAL DIRECTION:

ANALYZE A 1 FT STRIP BELOW THE STEM WHERE THE LOAD AS FOLLOWS:

DEAD = 1.90 KSF = 1.9 K/FT
LIVE = 117 K.FT / 3 = 39.0 K/FT (OVERTURNING)

TOTAL 40.9 K/FT

ASSUME MULTI-SPAN BEAM W/ NEGATIVE MOMENT OVER PILE HEADS = $.1 wL^2$

$M_{max} = .1(40.9)(5.75)^2 = 135.2$ K.FT

1. $M_u = 1.3(1.7)(135.2) = 298.$ K.FT

2. CAPACITY OF MEMBER BASED ON $\rho = .25\rho_b$

$\phi M_n = 268$ K.FT (P. 2)

3. $M_u / \phi M_n = 298/268 = 1.11$

4. THIS ANALYSIS IS CONSERVATIVE, ACTUAL SLAB LOAD DUE TO OVERTURNING WILL SPREAD TRANSVERSELY. ∴ OK

GATED MONOLITH STRUCTURAL DESIGN

I) GENERAL:

1. BASIC DESIGN CRITERIA:

$f'_c = 3000$ PSI

$f_y = 60000$ PSI

$\rho = .25 \rho_b$

ASSUME SINGLY REINFORCED SECTIONS

2. DESIGN MAJOR COMPONENTS:

A. SLAB

B. PIER

C. BACKWALL

D. OPERATING FLOOR

3. REINFORCING DETAILS TO BE PREPARED DURING P & S.

GATED MONOLITH G-1

I) SLAB DESIGN : USE PROGRAM "CFRAME" TO ANALYZE BOTH TRANSVERSE AND LONGITUDINAL STRIPS

1) LOADING : LOADS FROM THE PIER MUST BE USED WITH THE OTHER UNIFORM LOADS. USE CASE I.

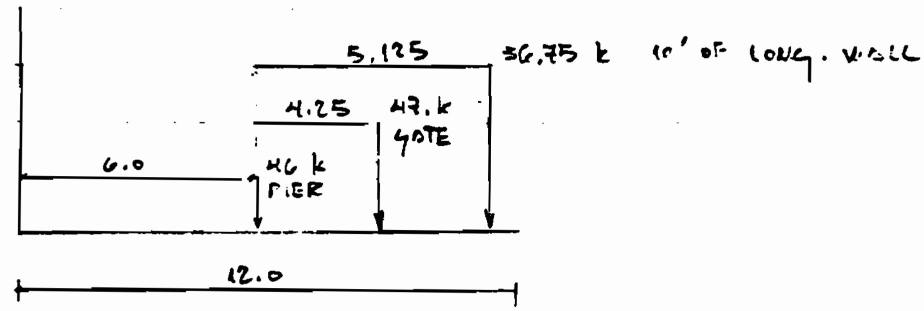
A) PIER LOADS : THE ACTUAL PIER FOOTPRINT ON THE SLAB IS COMPLEX AND SUBJECT TO REFINEMENT. FOR SIMPLIFICATION, ASSUME THE PIER IS 1.0' BY 12.0'. THE ACTUAL PIER WIDTH VARIES FROM 10" TO 1'6" AND THE LENGTH IS 12' 5", SO THE ASSUMED SECTION IS REPRESENTATIVE.

PIER PROPERTIES : $A = 12.0 \text{ FT}^2$

$S = bd^2/6 = 1(12)^2/6 = 24.0 \text{ FT}^3$

THE PRESSURE DISTRIBUTION OF LOAD TRANSFERRED FROM THE PIER TO THE SLAB WILL NOT BE UNIFORM DUE TO THE OVERTURNING EFFECTS OF BOTH THE DEAD AND LIVE LOADS ON THE PIER

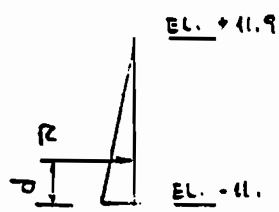
1) DEAD LOADS: ONE PIER => NON-OFFSET GATE



$V_D = 46 + 47 + 36.75 = 129.75$ k => 130. k

$M_D = 47(4.25) + 36.75(5.125) = 388.0$ k·FT

2) LIVE LOADS: ONE PIER



$R = .5(.0625)(22.9)^2 = 16.3$ k (PER FT)

$d = 22.9/3 = 7.63'$

CENTER PIER: 10' OF GATE PLUS 1.5' PIER WIDTH

$M_L = 11.5(16.3)(7.6) = 1425.$ k·FT

EDGE PIER: 5' OF GATE PLUS 1.5' PIER WIDTH

$M_L = 6.5(16.3)(7.6) = 805.$ k·FT

3) TOTAL LOADS:

$V_T = V_D + V_L = 130. + 0. = 130.$ k

CENTER PIER:

$M_T = M_D + M_L = 388 + 1425 = 1813$ k·FT

EDGE PIER

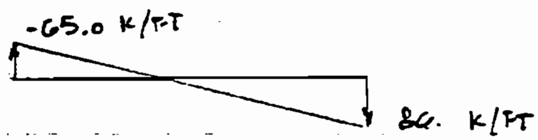
$M_T = M_D + M_L = 388 + 805 = 1193$ k·FT

4) PRESSURE DISTRIBUTION OF BASE OF PIER.
SINCE PIER IS 1.0' WIDTH, PRESSURE VALUES (KSF) WILL EQUAL LINEAR LOAD VALUES (K/FT).

CENTER PIER: $V_T / A \pm M_T / S$

$130/12. \pm 1813/24.$

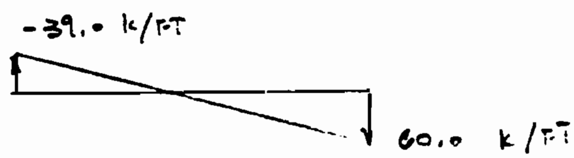
10.8 ± 75.5



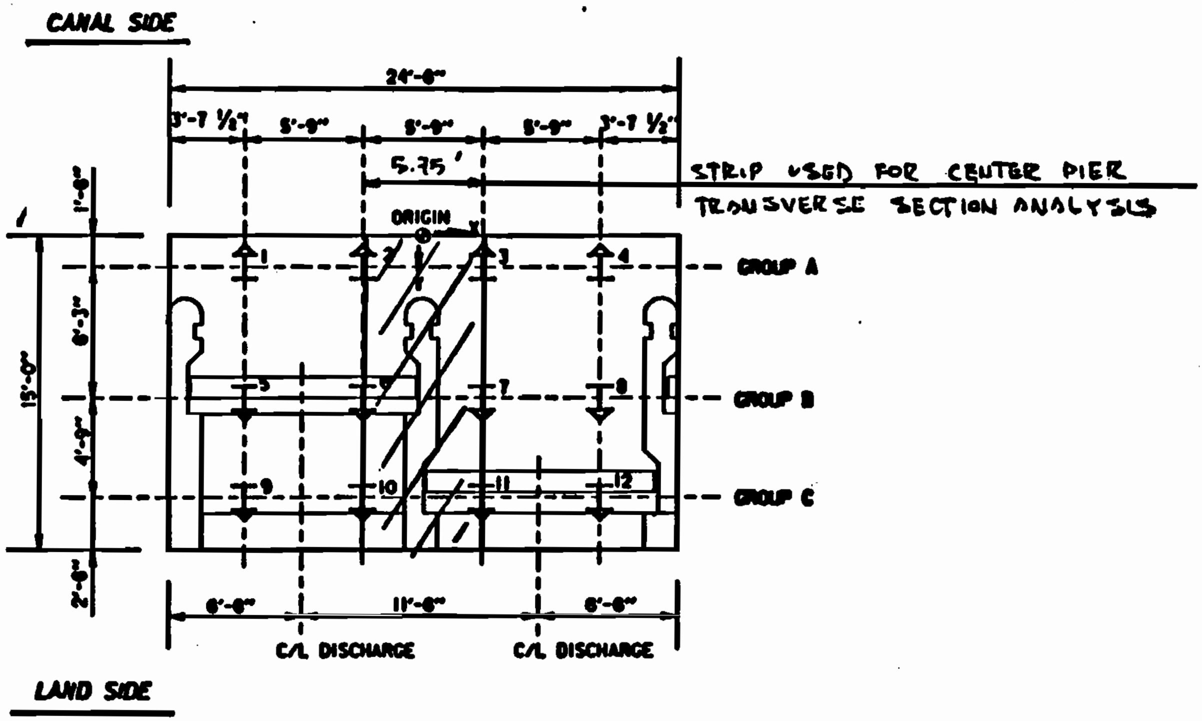
EDGE PIER: $V_T / A \pm M_T / S$

$130/12 \pm 1193/24.$

10.8 ± 49.7



USE THESE LOADS FOR ANALYZING SLAB STRIPS.



PILE LEGEND

- 6V on 1H BATTER - GROUP A - HP14 X 73 (ASTM A36) x 88 FT.
- 12V on 1H BATTER - GROUP B - HP14 X 73 (ASTM A36) X 88 FT.
- 10V on 1H BATTER - GROUP C - HP14 X 73 (ASTM A36) X 88 FT.

PILE LAYOUT - MONOLITH G-1

SCALE: 1/4" = 1'- 0"

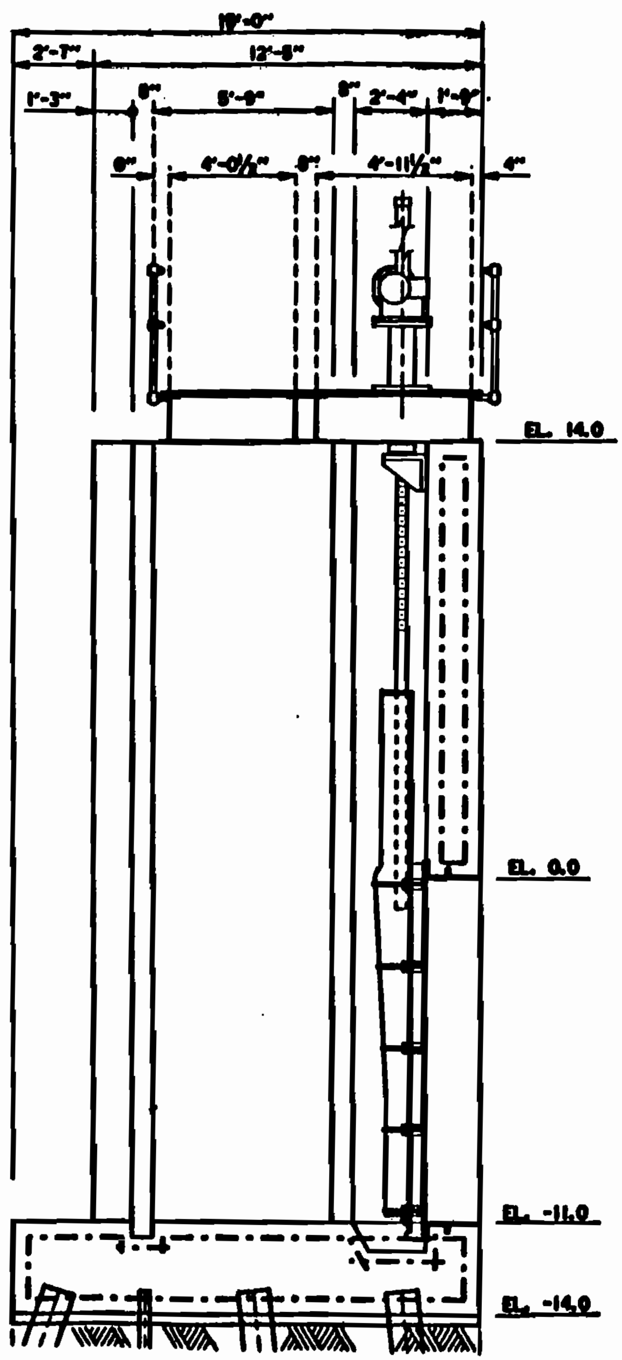
12" 0 5' 10' 15' 20'

LOAD CASES

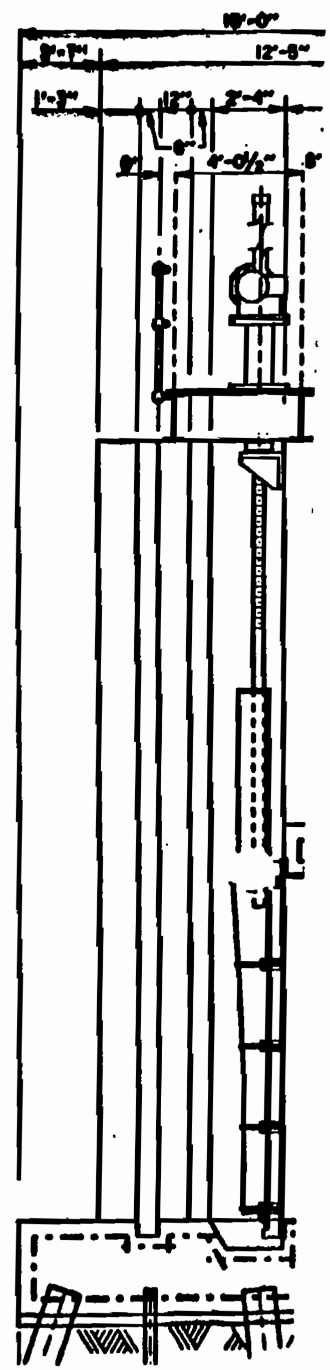
CASE I - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, IMPERVIOUS SHEETPILE CUT-OFF.

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	[illegible]
I	0.	343.0	641.0	9093.0	-504.0	[illegible]
II	0.	343.0	493.0	7939.0	-504.0	[illegible]
III *	0.	322.0	506.0	7876.0	-404.0	[illegible]



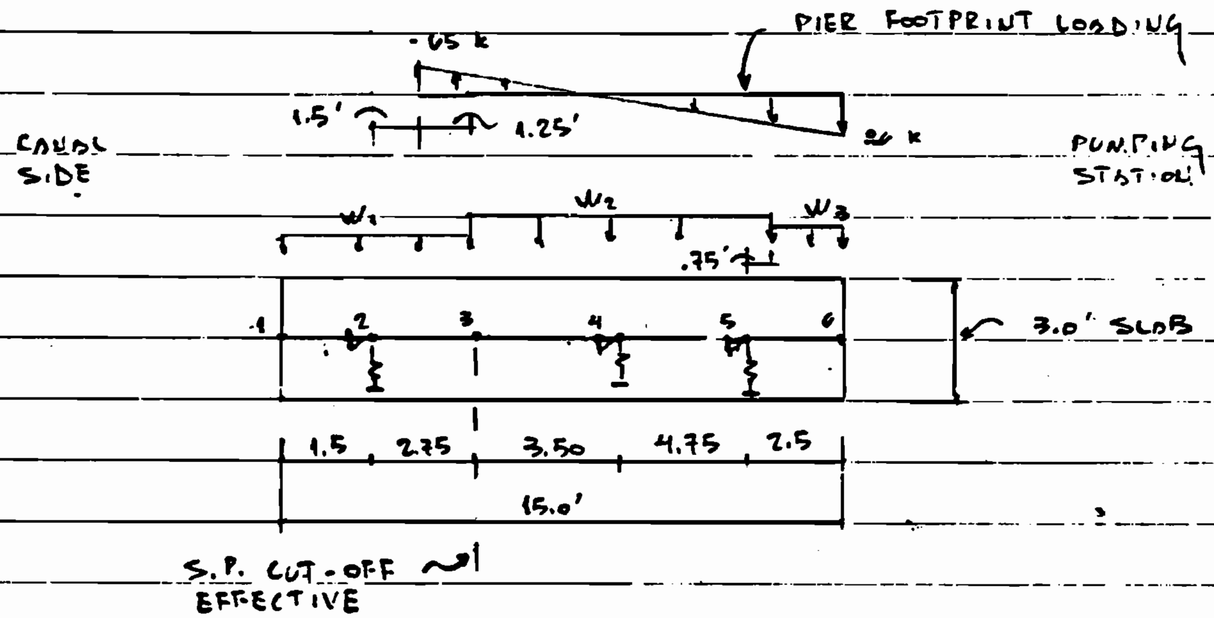
2'-7"
12'-5"
1'-3"
5'-9"
2'-4"
4'-0½"
4'-11½"
4"
EL. 14.0
EL. 0.0
EL. -11.0
EL. -14.0



12'-5"
1'-3"
2'-4"
4'-0½"

CFRAME MODEL OF G-1 SLAB

1. TRANSVERSE DIRECTION ; LOAD CASE 1 , NON-OFFSET GATE ADJACENT TO PIER , 5.75 FT STRIP W/ VERTICAL PILE SPRINGS.



	CONCRETE	STILL WATER	UPLIFT		STRIP WIDTH	
w_1 =	.45	1.43	-.45	: .28	(5.75)	= 1.61 k/FT
w_2 =	.45	1.43	-.50	: 1.38	(5.75)	= 7.93 k/FT
w_3 =	.45	1.19	-.50	: 1.14	(5.75)	: 6.56 k/FT

DUE TO WATER TRAPPED IN CULVERT

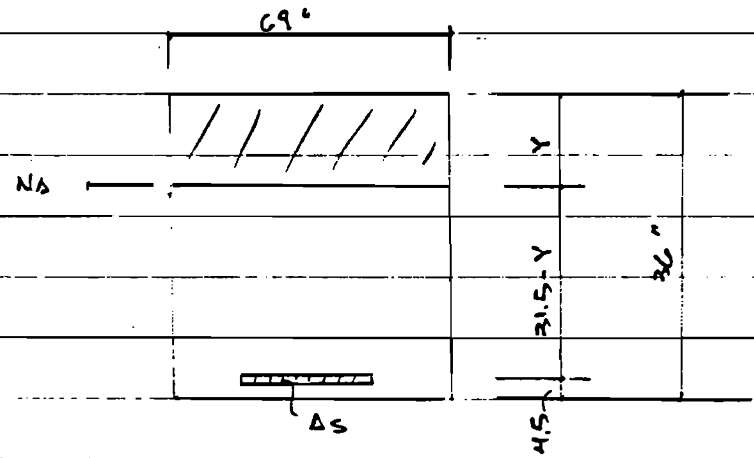
VERTICAL PILE SPRING CONSTANT : CAE/L_E (CPGA manual)

C = 2.0 FRICTION PILE

A = 21.4 , E = 29000 KSI , L_E = 88 FT = 1056"

B33 = 2(21.4)(29000)/1056 = 1175. k/IN

MEMBER PROPERTIES: CRACKED TRANSFORMED SECTION



USE $A = \rho bd = .25\rho_b\, bd$

$$\rho_b = \frac{.85\,\beta_1 f'_c\, 87}{f_y(87+f_y)} = \frac{.85^2(3)(87)}{60(87+60)} = .0214$$

$$A = .25(.0214)(69)(31.5) = 11.63 \text{ in}^2$$

$$n = E_s/E_c = 29000000 \,/\, 57000\sqrt{3000} = 9.28 \Rightarrow 9$$

FIND N.A.: $Y(69)(Y/2) = nA_s(31.5-Y)$

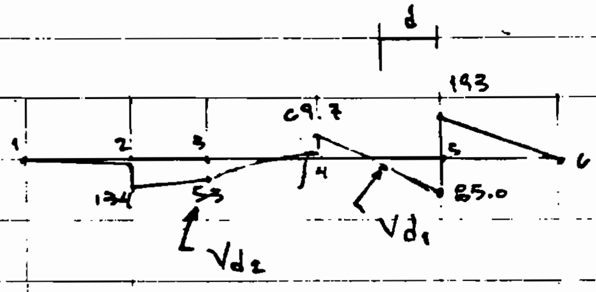
$$34.5Y^2 = 9(11.63)(31.5-Y)$$
$$34.5Y^2 = 3297.1 - 104.6Y$$
$$34.5Y^2 + 104.6Y - 3297.1 = 0$$
$$Y = 8.3''$$

$$I_{cr} = bY^3/3 + nA_s(31.5-Y)^2$$
$$= 69(8.3)^3/3 + 9(11.63)(31.5-8.3)^2$$
$$= 13151 + 56337 = 69488 \text{ in}^4$$

A = AXIAL AREA = USE C9(36) = 2484 IN^2

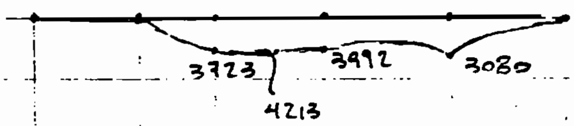
AS = SHEAR AREA = USE C9(83) = 572.7 IN^2

SHEAR: k



FOR 5.75' STRIP

MOMENT: K·FT



FLEXURE : BASED ON 1 FT STRIP

1. MEMBER CAPACITY BASED ON $\rho = .25\rho_b$

$$A_s = .25(.0214)(12)(31.5) = 2.02 \text{ IN}^2$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$a = A_s f_y / .85 f'_c b = 2.02(60)/.85(3)(12) = 3.96''$$

$$\phi M_n = .9(2.02)(60)[31.5 - 3.96/2] = 3220 \text{ K·IN}$$

2. $M_u = 1.3(1.7)(4213/5.75) = 1619.$ K·IN

3. $\phi M_n > M_u$ SECTION IS ADEQUATE ✓

SHEAR :

1. FLEXURAL SHEAR CRACKING

CHECK IF $V_u > \phi V_c$

2. $\phi V_c = \phi\, 2\sqrt{f'_c}\, b d = .85(2)(3000)^{1/2}(12)(31.5) = 35196$ LB

3. $V_{d1} = -(31.5)(85+70)/[4.75(12)] + 85 = -0.6$ k

$V_{d2} \approx 53$ k

$V_u = 1.3(1.7)(53/5.75) = 20.3$ k

4. $\phi V_c > V_u$ NO SHEAR REINFORCEMENT

```
010 G1 SLAB; 2 SPAN; TRANSVERSE DIRECTION; 5.75 FT STRIP @ CENTER
020 KSI FT IN IN KIP
030 6 5 2 3000 0.15
040 1 0. 0., 2 1.5  0., 3 4.25 0., 4 7.75 0., 5 12.5 0., 6 15. 0.
050 FIX X 2 4 5
060 FIX KY 1175 2 4 5
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6
130 69488 2484 573 1 2 3 4 5
150 LOAD CASE 1 0 6 0 0 0  GRAVITY  LOADS
160 0.0    1.59  1.50   1.59  0.  1
165 0.0    1.59  2.75   1.59  0.  2
170 0.0    7.93  3.50   7.93  0.  3
180 0.0    7.93  4.75   7.93  0.  4
185 0.0    7.93  0.75   7.93  0.  5
190 0.75   6.50  2.50   6.50  0.  5
200 LOAD CASE 2 0 4 0 0 0  BENDING LOADS
210 1.25   -65.0  2.75  -49.3  0.  2
215 0.     -49.3  3.50   -5.2  0.  3
220 0.     -5.2   4.75   54.5  0.  4
225 0.     54.5   2.50   86.0  0.  5
230 COMB 1  1 1., 2 1.  COMB1
```

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1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
 PROGRAM CFRAME V02.05 24JUL84
 *-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
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RUN DATE = 95/02/03
RUN TIME = 8.07.04

G1 SLAB; 2 SPAN; TRANSVERSE DIRECTION; 5.75 FT STRIP @ CENTER

*** JOINT DATA ***

JOINT	X	Y	FIXITY X	FIXITY Y	FIXITY R	KX	KY	KR
	--- FT ---					---KIP / IN---		IN-KIP/RAD
1	.00	.00						
2	1.50	.00	*				.118E+04	
3	4.25	.00						
4	7.75	.00	*				.118E+04	
5	12.50	.00	*				.118E+04	
6	15.00	.00						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	1.50	.6949E+05	.2484E+04	.5730E+03	.3000E+04	.1304E+04
2	2	3	2.75	.6949E+05	.2484E+04	.5730E+03	.3000E+04	.1304E+04
3	3	4	3.50	.6949E+05	.2484E+04	.5730E+03	.3000E+04	.1304E+04
4	4	5	4.75	.6949E+05	.2484E+04	.5730E+03	.3000E+04	.1304E+04
5	5	6	2.50	.6949E+05	.2484E+04	.5730E+03	.3000E+04	.1304E+04

*** LOAD CASE 1 GRAVITY LOADS

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.1590E+01	1.50	.1590E+01	.00
2	.00	.1590E+01	2.75	.1590E+01	.00
3	.00	.7930E+01	3.50	.7930E+01	.00
4	.00	.7930E+01	4.75	.7930E+01	.00
5	.00	.7930E+01	.75	.7930E+01	.00
5	.75	.6500E+01	2.50	.6500E+01	.00

*** LOAD CASE 2 BENDING LOADS

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
2	1.25	-.6500E+02	2.75	-.4930E+02	.00

3	.00	-.4930E+02	3.50	-.5200E+01	.00
4	.00	-.5200E+01	4.75	.5450E+02	.00
5	.00	.5450E+02	2.50	.8600E+02	.00

*** LOAD CASE COMBINATIONS ***

LOAD CASE	LOAD CASE FACTORS 1	2
1	1.00	1.00

1 LOAD CASE 1 COMB1

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	.0000E+00	.1435E+00	-.1728E-02
2	.0000E+00	.1124E+00	-.1729E-02
3	.0000E+00	.5666E-01	-.2063E-02
4	.0000E+00	-.4699E-01	-.2888E-02
5	.0000E+00	-.2366E+00	-.3654E-02
6	.0000E+00	-.3537E+00	-.3806E-02

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
	2	.0000E+00	.2385E+01	-.2147E+02	-.2147E+02	18.00
2	2	.0000E+00	-.1344E+03	-.2147E+02	-.2147E+02	.00
	3	.0000E+00	.5308E+02	-.3723E+04	-.3723E+04	33.00
3	3	.0000E+00	-.5308E+02	-.3723E+04	-.3723E+04	.00
	4	.0000E+00	-.1454E+02	-.3992E+04	-.4213E+04	21.00
4	4	.0000E+00	.6975E+02	-.3992E+04	-.2304E+04	37.62
	5	.0000E+00	.8500E+02	-.3080E+04	-.3992E+04	.00
5	5	.0000E+00	.1929E+03	-.3080E+04	.0000E+00	30.00
	6	.0000E+00	.0000E+00	.0000E+00	-.3080E+04	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.0000E+00	-.1320E+03	.0000E+00
4	.0000E+00	.5521E+02	.0000E+00
5	.0000E+00	.2779E+03	.0000E+00
TOTAL	.0000E+00	.2011E+03	

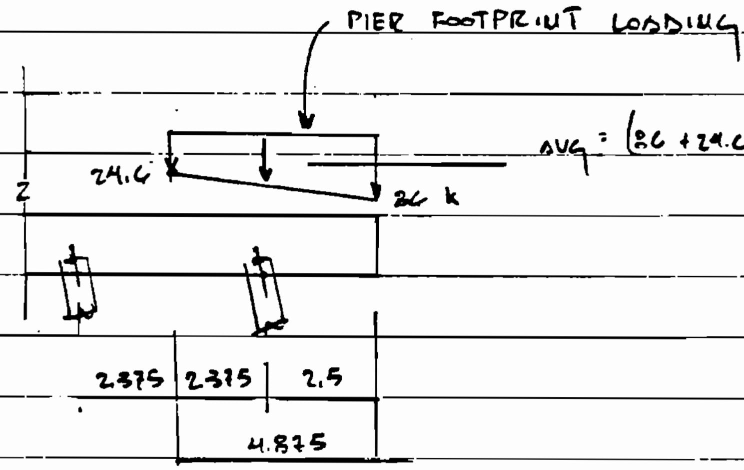
1 MEMBER END FORCES

LOAD MOMENT

MEMBER	CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	EXTREMA IN-KIP	LOCATION IN
1	1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
		2	.0000E+00	.2385E+01	-.2147E+02	-.2147E+02	18.00
2	1	2	.0000E+00	-.1344E+03	-.2147E+02	-.2147E+02	.00
		3	.0000E+00	.5308E+02	-.3723E+04	-.3723E+04	33.00
3	1	3	.0000E+00	-.5308E+02	-.3723E+04	-.3723E+04	.00
		4	.0000E+00	-.1454E+02	-.3992E+04	-.4213E+04	21.00
4	1	4	.0000E+00	.6975E+02	-.3992E+04	-.2304E+04	37.62
		5	.0000E+00	.8500E+02	-.3080E+04	-.3992E+04	.00
5	1	5	.0000E+00	.1929E+03	-.3080E+04	.0000E+00	30.00
		6	.0000E+00	.0000E+00	.0000E+00	-.3080E+04	.00

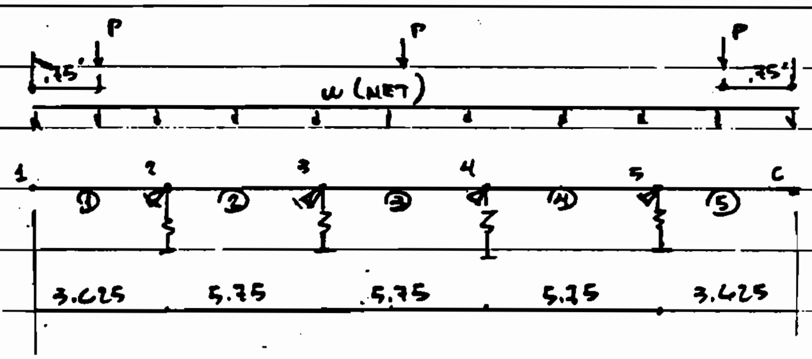
2. LONGITUDINAL DIRECTION: LOAD CASE 1, 4.875' STRIP AS SHOWN:

CANAL SIDE



AVG = (86 + 24.6)/2 = 55.3 k

ANALYZE 3 SPANS w/ OVERHANGS; VERTICAL PILE SPRINGS

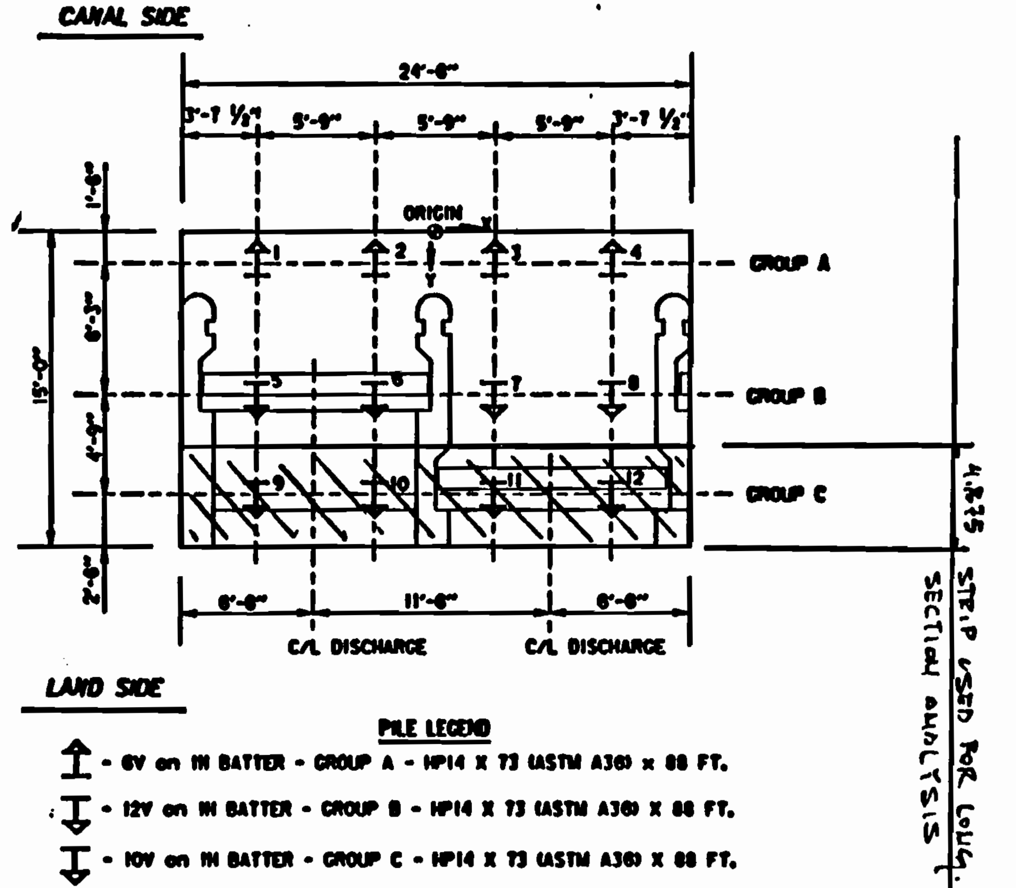


P = PIER LOAD = 55 k (SEE ABOVE)

w (NET) : CONCRETE .45 STILL WATER 1.43 UPLIFT -.50

: 1.38 (4.875) = 6.72 k/FT

5 | 4 | 3



PILE LEGEND

- 6V on 1H BATTER - GROUP A - HP14 X 73 (ASTM A36) x 88 FT.
- 12V on 1H BATTER - GROUP B - HP14 X 73 (ASTM A36) X 88 FT.
- 10V on 1H BATTER - GROUP C - HP14 X 73 (ASTM A36) X 88 FT.

PILE LAYOUT - MONOLITH G-1

SCALE: 1/4" = 1'- 0"

12" 0 5' 10' 15' 20'

LOAD CASES

CASE I - GATE CLOSED, CANAL SWL EL. 11.9 NGVD, SWL INSIDE DISCHARGE CULVERTS EL. 8.0 NGVD, STORM WIND LOAD, IMPERVIOUS SHEETPILE CUT-OFF.

CASE II - GATE CLOSED, CANAL [illegible]

APPLIED LOADS

LOAD CASE	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kips-ft)	My (kips-ft)	(k[illegible]
I	0.	343.0	641.0	9093.0	-504.0	15
II	0.	343.0	493.0	7939.0	-504.0	15
III *	0.	322.0	506.0	7876.0	-404.0	[illegible]
IV *	0.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

PILE SPRINGS = AS BEFORE

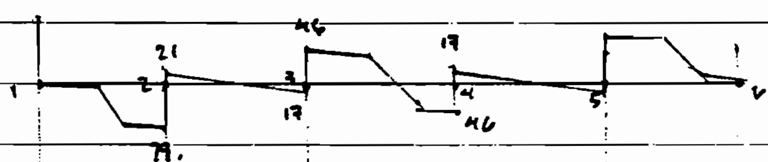
MEMBER PROPERTIES:

USE RATIO: $I_{CR} = 69483\,(4.375/5.15) = 58913.$

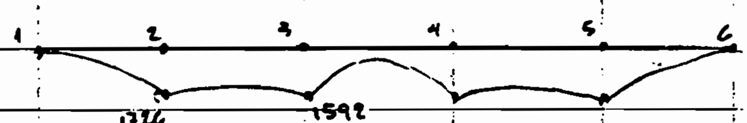
$A = 4.375\,(12)(36) = 2106$

$AS = 4.375\,(12)(8.3) = 436$

SHEAR:



MOMENT:



FLEXURE: BASED ON 1-FT STRIP

1. MEMBER CAPACITY BASED ON $\rho = .25 \rho_b$

$\phi M_n = 3220$ K-IN (P. 8)

2. $M_u = 1.3(1.7)(1726/4.875) = 782$ K-IN

3. $\phi M_n > M_u \Rightarrow$ SECTION IS ADEQUATE

SHEAR: FLEXURAL SHEAR CRACKING => NO

1. PUNCHING SHEAR CAPACITY:

$d/2 = 31.5/2 = 15.75''$
$b_o = 4(14 + 15.75) = 119.''$

$\phi V_c = \phi 4 \sqrt{f'_c}\, b_o d$
$= .85(4)(3000)^{1/2}(119)(31.5) = 698066$ LB
$= 698$ K

2. $V_u = 1.3(1.7)(79 + 21) = 221$ K

3. $\phi V_c > V_u$ OK

```
010 G1 SLAB; LONGITUDINAL DIRECTION; 3 SPANS W/ OVERHANGS; 4.875 STRIP
020 KSI FT IN IN KIP
030 6 5 1 3000 0.15
040 1 0 0,2 3.625 0,3 9.375 0,4 15.125 0,5 20.785 0,6 24.5 0
050 FIX X 2 3 4 5
060 FIX KY 1175 2 3 4 5
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6
130 58913 2106 486 1 2 3 4 5
150 LOAD CASE 1 0 5 0 0 0  NET LOADS
160 0.0      6.72  3.625  6.72  0.  1 5
170 0.0      6.72  5.750  6.72  0.  2 3 4
180 1.3125   55.0  2.3125 55.0  0.  1
190 2.375    55.0  3.375  55.0  0.  3
200 1.3125   55.0  2.3125 55.0  0.  5
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1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
PROGRAM CFRAME V02.05 24JUL84
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RUN DATE = 95/02/03
RUN TIME = 8.52.27

G1 SLAB; LONGITUDINAL DIRECTION; 3 SPANS W/ OVERHANGS; 4.875 STRIP

*** JOINT DATA ***

JOINT	X	Y	X	Y	R	KX	KY	KR
	--- FT ---		FIXITY			---KIP / IN---		IN-KIP/RAD
1	.00	.00						
2	3.63	.00	*				.118E+04	
3	9.38	.00	*				.118E+04	
4	15.13	.00	*				.118E+04	
5	20.78	.00	*				.118E+04	
6	24.50	.00						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	3.63	.5891E+05	.2106E+04	.4860E+03	.3000E+04	.1304E+04
2	2	3	5.75	.5891E+05	.2106E+04	.4860E+03	.3000E+04	.1304E+04
3	3	4	5.75	.5891E+05	.2106E+04	.4860E+03	.3000E+04	.1304E+04
4	4	5	5.66	.5891E+05	.2106E+04	.4860E+03	.3000E+04	.1304E+04
5	5	6	3.72	.5891E+05	.2106E+04	.4860E+03	.3000E+04	.1304E+04

*** LOAD CASE 1 NET LOADS

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.6720E+01	3.63	.6720E+01	.00
1	1.31	.5500E+02	2.31	.5500E+02	.00
2	.00	.6720E+01	5.75	.6720E+01	.00
3	.00	.6720E+01	5.75	.6720E+01	.00
3	2.38	.5500E+02	3.38	.5500E+02	.00
4	.00	.6720E+01	5.66	.6720E+01	.00
5	.00	.6720E+01	3.63	.6720E+01	.00
5	1.31	.5500E+02	2.31	.5500E+02	.00

1 LOAD CASE 1 NET LOADS

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	.0000E+00	-.1245E+00	.8566E-03
2	.0000E+00	-.8559E-01	.7377E-03
3	.0000E+00	-.5457E-01	.1760E-03
4	.0000E+00	-.5463E-01	-.1788E-03
5	.0000E+00	-.8524E-01	-.7361E-03
6	.0000E+00	-.1250E+00	-.8550E-03

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
	2	.0000E+00	.7936E+02	-.1726E+04	-.1726E+04	43.50
2	2	.0000E+00	.2121E+02	-.1726E+04	-.1325E+04	37.26
	3	.0000E+00	.1743E+02	-.1596E+04	-.1726E+04	.00
3	3	.0000E+00	.4669E+02	-.1596E+04	-.4009E+03	34.50
	4	.0000E+00	.4695E+02	-.1605E+04	-.1605E+04	69.00
4	4	.0000E+00	.1724E+02	-.1605E+04	-.1340E+04	31.24
	5	.0000E+00	.2080E+02	-.1726E+04	-.1726E+04	67.92
5	5	.0000E+00	.7936E+02	-.1726E+04	.0000E+00	44.58
	6	.0000E+00	.0000E+00	.0000E+00	-.1726E+04	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.0000E+00	.1006E+03	.0000E+00
3	.0000E+00	.6412E+02	.0000E+00
4	.0000E+00	.6419E+02	.0000E+00
5	.0000E+00	.1002E+03	.0000E+00
TOTAL	.0000E+00	.3290E+03	

1 MEMBER END FORCES

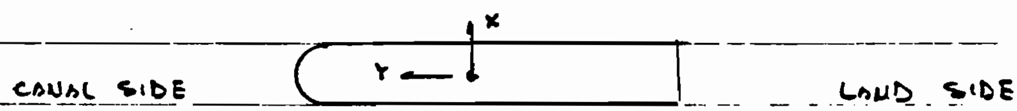
MEMBER	LOAD CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	1	.0000E+00	.0000E+00	.0000E+00	.0000E+00	.00
		2	.0000E+00	.7936E+02	-.1726E+04	-.1726E+04	43.50
2	1	2	.0000E+00	.2121E+02	-.1726E+04	-.1325E+04	37.26
		3	.0000E+00	.1743E+02	-.1596E+04	-.1726E+04	.00
3	1	3	.0000E+00	.4669E+02	-.1596E+04	-.4009E+03	34.50
		4	.0000E+00	.4695E+02	-.1605E+04	-.1605E+04	69.00
4	1	4	.0000E+00	.1724E+02	-.1605E+04	-.1340E+04	31.24
		5	.0000E+00	.2080E+02	-.1726E+04	-.1726E+04	67.92
5	1	5	.0000E+00	.7936E+02	-.1726E+04	.0000E+00	44.58
		6	.0000E+00	.0000E+00	.0000E+00	-.1726E+04	.00

III) PIER DESIGN :

1. GENERAL :

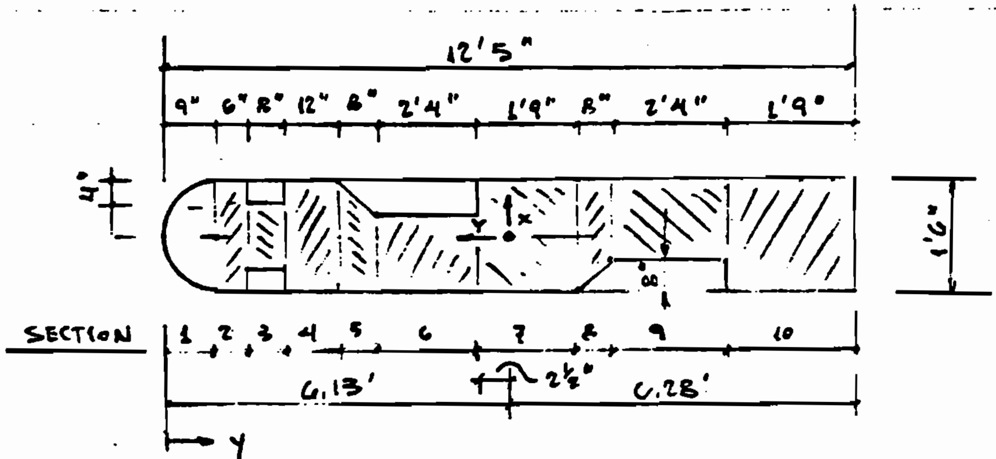
A. PIER IS LIKE A WALL SPANNING BETWEEN THE BASE SLAB AND OPERATING FLOOR, w/ INTERMEDIATE SUPPORT AT THE OFF-SET GATE SEATING SLAB.

B. ASSUME PIER ORIENTATION FOR ANALYSIS AS FOLLOWS :



C. LOADING : X-AXIS BENDING => LOAD CASE 1

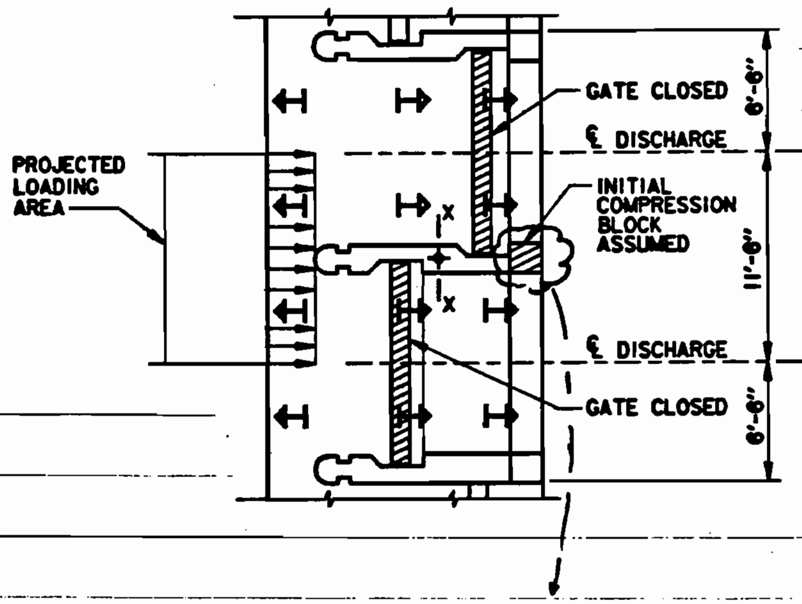
PLAN @ BASE OF PIER



SECTION	AREA (FT^2)	$\bar{y}$	AREA ($\bar{y}$)	d	Ad^2
1	$.5(\pi)(.75)^2 = .88$	0.45	.39	5.68	28.4
2	$.5(1.5) = .75$	1.00	.75	5.13	19.7
3	$\frac{8}{12}(1.5) = 1.00$	1.58	1.58	4.55	20.7
4	1.5	2.42	3.63	3.71	20.6
5	$\frac{8}{12}(1.5) - .5(\frac{8}{12})^2 = .78$	3.25	2.53	2.88	6.5
6	$2.33(\frac{10}{12}) = 1.94$	4.75	9.21	1.38	3.7
7	$1.75(1.5) = 2.62$	6.79	17.79	0.66	1.1
8	.78	8.00	6.24	1.87	2.7
9	1.94	9.50	18.43	3.37	22.0
10	2.82	11.55	30.26	5.42	76.9
	14.81 FT^2		90.81 FT^3		202.3 FT^4 = I

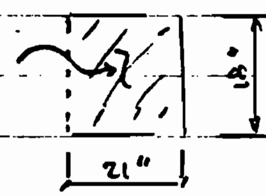
N.A. = 90.81 / 14.81 = 6.13' = 6' 1½" = 73½"

2) FLEXURE : X-AXIS BENDING



SECTION 10 OF PIER : SEE P. 14

$A_c = 378 \text{ in}^2$



A. USE ITERATIVE METHOD TO FIND REINFORCING BASED ON M_u.

1. $M_x = [\,510.6(8.6-3) - 185.8(7.73-3) + 90(14.125-7.89)$
$+ 94(10.875-7.92)\,]/3$
$= [\,2859.4 - 878.8 + 558.4 + 277.8\,]/3 = 938.9$ K.FT

$M_{ux} = 1.3(1.7)(938.9) = 2075$ K.FT

WITH COMPRESSION BLOCK AS SHOWN => $a = 21''$

$\phi M_{nx} = \phi T (d - a/2)$ MUST EQUAL M_{ux}

USE $d = 12'5'' - 5'' = 12' = 144''$

$2075(12) = .9T(144 - 21/2)$

$24900 = 120.1T \Rightarrow T = 207.2 \text{ k}$

$A_s \approx T/f_y = 207/60 = 3.45 \text{ IN}^2$

RECOMPUTE a BY $C_c = T$

$C_c = .85 f'_c A_c = .85(3) A_c = 207.2$

$A_c = 81.2 \text{ IN}^2 < 378$ ASSUMED INITIALY

USE $ab = A_c \Rightarrow a(18) = 81.2 \Rightarrow a = 4.51''$

RECOMPUTE A_s W/ REVISED a

$24900 = .9T(144 - 4.51/2) \Rightarrow T = 195.2 \text{ k}$

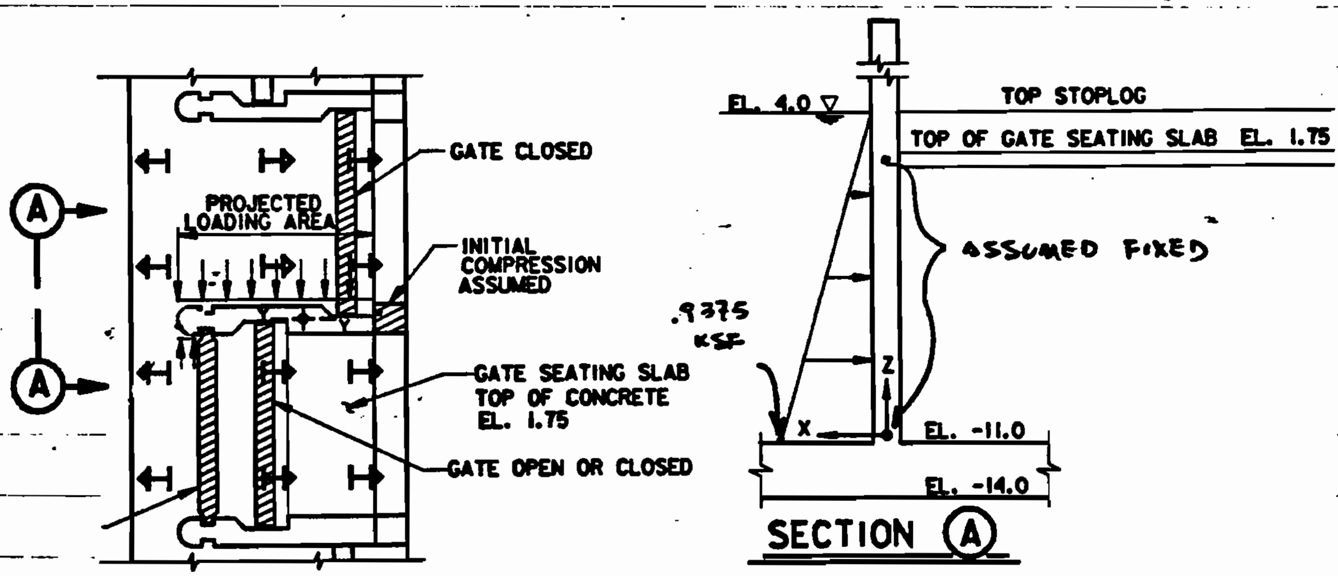
$A_s \approx 195.2/60 = 3.25 \text{ IN}^2$

B) CHECK MINIMUM STEEL BASED ON GROSS PIER AREA

$$A_{s\,MIN} = 200\,bd/f_y = 200(12)(144)/60 = 8.64\ \text{IN}^2$$

C. USE MINIMUM STEEL REQ'MT. DISTRIBUTE STEEL AROUND PERIMETER OF PIER. INSURE THAT A MINIMUM OF 3.25 IN^2 IS LOCATED IN SECTIONS 1-4 (P. 14), THE ANTICIPATED PRIMARY TENSION ZONE FOR X-AXIS BENDING.

3. FLEXURAL: Y-AXIS BENDING => LOAD CASE 5
DEWATERED CHAMBER



ONE-FT STRIP :

1. $M_{max} = wL^2/20 = [15(.0625)](15)^2/20 = 10.54$ K·FT

$M_u = 1.3(1.7)(10.54) = 31.91$ K·FT

2. CAPACITY OF 18" THICK COLUMN BASED ON $\rho = .25\rho_b$

$A_s = .25(.0214)(12)(18-3) = .963$ IN^2

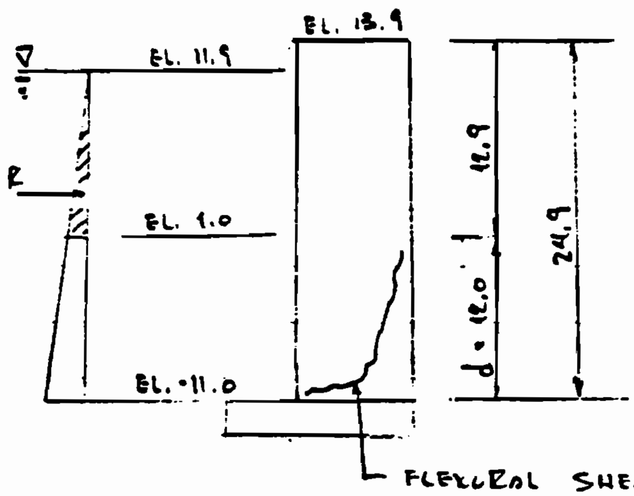
$a = .963(60)/.85(3)(12) = 1.89$

$\phi M_n = .9(.963)(60)[15-.94] = 730.9$ K.IN $= 60.9$ K.FT

3. $\phi M_n > M_u$ SECTION IS ADEQUATE

3) SHEAR R/S: Y-AXIS LOADING

1) ASSUME PIER ACTS LIKE A CANTILEVER BEAM AND DEVELOPS FULL AREA STRENGTH AT BASE



$R = .5(.0625)(10.9)^2 = 3.71$ k

FOR TYPICAL BEAM BEHAVIOR, A FLEXURAL SHEAR CRACK TAKES A DISTANCE d TO DEVELOP, SO SHEAR REINFORCEMENT REQ'MTS ARE CHECKED AT d FROM BEAM END.

FOR $d = 12.0'$:   10' OF LONG. WALL PLUS 1.5' PIER

$V_u = 1.3(1.7)(3.71)(11.5) = 94.3$ k

$\phi V_c = \phi 2\sqrt{f'_c}\,A_c = .85(2)\sqrt{3000}\,(12)(144)/1000 = 160.8$ k

∴ NO SHEAR REINFORCEMENT REQ'D.

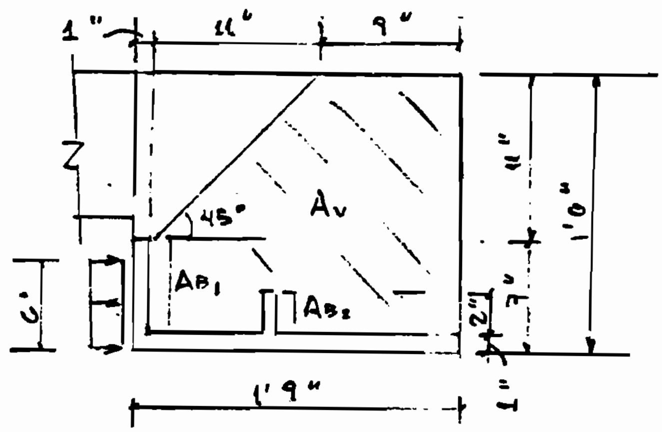
ANALYZING SHEAR OVER THE ENTIRE PIER AREA IS UNREALISTIC. CHECK LOCAL CONDITIONS. SEE PART B.

B) PIER SHEAR R/S (CONT.):

2) DESIGN SECTIONS 7 AND 10 (SEE PLAN @ BASE OF PIER) TO TAKE SHEAR LOAD FROM GATES.

"F" SHAPE THIMBLE PROVIDED BY GATE MANUFACTURER (FOR THIS CASE => RODNEY HUNT, INC) IS DESIGNED TO SPREAD SHEAR LOAD OVER ADEQUATE CONCRETE BEARING AND SHEAR AREA.

LOAD FROM GATE GUIDE



AS A CHECK, ANALYZE SECTION @ PIER BASE WHERE GREATEST SHEAR LOAD EXISTS.

½ GATE WIDTH

LOAD FROM GATE GUIDE : $.0625(22.9)(5') = 7.1$ k

FACTORED LOAD $= 1.3(1.7)(7.1) = 15.7$ k

$A_{B1} = 6(12) = 72.0$ in^2

$A_{B2} = 2(12) = 24.0$ in^2

> $A_{BT} = 72 + 24 = 96.$ in^2

$f_B = 15.7/96 = 0.163$ ksi

$F_B = .35 f'_c = .35(3) = 1.05$ ksi ✓

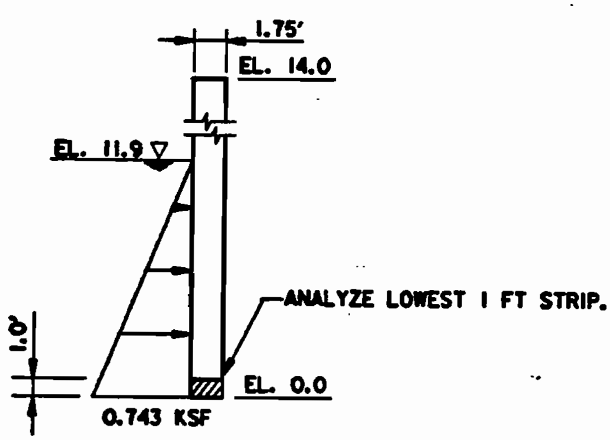
$A_V = (1.41c)^2 - .5(.91)^2 = 1.59 \text{ FT}^2 = 229. \text{ IN}^2$

$\phi V_c = \phi 2 \sqrt{f'_c}\, A_v = .85(2)\sqrt{3000}\,(229)/1000 = 21.3 \text{ k}$ ✓

CONCRETE STRENGTH ALONE IS ADEQUATE.
"F" THIMBLE PREVENTS DEVELOPMENT OF
SHEAR CRACKING ANYWAY.

∴ NO SHEAR REINFORCEMENT REQ'D.

III) LONGITUDINAL WALL DESIGN LOAD CASE 1



WALL SPAN = 10.' ; $M_{MAX} = wL^2/12 = .743(10^2)/12 = 6.19$ k·FT

1. $M_u = 1.3(1.7)(6.2) = 13.7$ k·FT

2. CAPACITY OF WALL BASED ON $\rho = .25\rho_b$

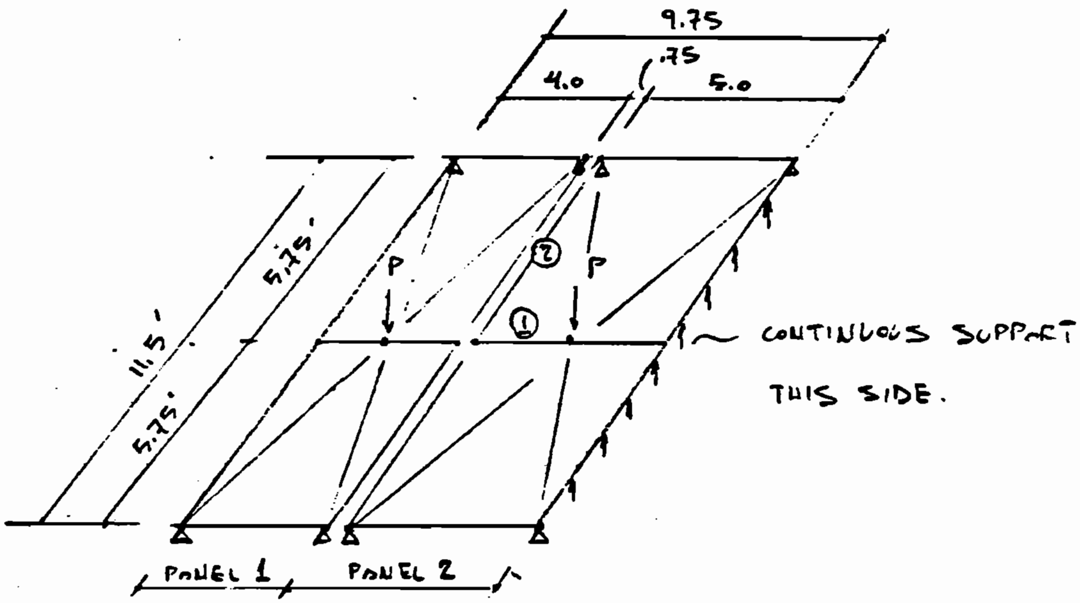
$A_s = .25(.0214)(12)(21-3) = 1.15 \text{ IN}^2$

$a = 1.15(60) / .85(3)(12) = 2.26$

$\phi M_n = .9(1.15)(60)[18 - 1.13] = 1047.$ K·IN $= 87.2$ K·FT

3. $\phi M_n > M_u$ SECTION IS ADEQUATE.

OPERATING FLOOR DESIGN :



IN - PLACE

GATE LOAD , P, ACTS ON EITHER PANEL 1 OR PANEL 2 , DEPENDING ON THE GATE BAY .

FROM: PREVIOUS ANALYSIS , PRELIM. DESIGN VOL. 1, "CLOSING GATE CONDITION W/ JAMMED GATE AND OPERATOR MOTOR REACHING LOCK ROTOR TORQUE GOVERNS MEMBER SIZES.

FROM GEN. ENG. , USE $P = 115\ k$

MEMBER (2) :

LOAD @ MID-SPAN = 115/2 = 57.5 k

M_u = (1.7)[57.5 (11.5)/4] = 281. k·FT

ASD: ASSUME $L_b < L_c$, ∴ $F_b = .66 F_y$

PER EM 1110-2-2101, USE $F_b = .83(.66)(F_y)$

FOR ASTM A36 ; $F_b = .83(.66)(36) = 19.7$ KSI

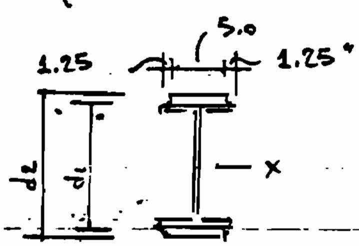
$S_{x\ REQ'D} = 281.(12)/19.7 = 171.0\ IN^3$

FOR ASTM A572 GRADE 50, $F_b = .83(.66)(50) = 27.4$ KSI

$S_{x\ REQ'D} = 281.(12)/27.4 = 123.1\ IN^3$

SOLUTION :

A) ADD FLANGE PLATES TO W18x50 (S_x = 88.9 IN^3) AS REQUIRED FOR STRENGTH AT MID-SPAN



TRY ½ PL : d_1 = 18.", d_2 = 19."

ADDITIONAL $I_x = 5(19^3 - 18^3)/12 = 428.\ IN^4$

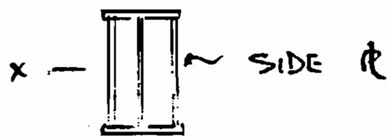
I_{TOTAL} = 800 + 428 = 1228. IN^4

$S_{x\ TOTAL}$ = 1228 / 9.5 = 130. IN^3

W/ 1.33 OVERSTRESS FACTOR:

$$S_x = 130(1.33) = 172.\ \text{IN}^3 = S_{x\ \text{REQ'D}}\ (A36)\ \checkmark$$

B) BOX-IN SIDES OF W18x50



TRY 3/8" PL:

$$\text{ADDITIONAL } I_x = 2\left(\tfrac{3}{8}\right)(16)^3/12 = 256\ \text{IN}^4 < 428\ \text{IN}^4$$

NO GOOD

TRY 1/2" PL:

$$I_x = 2\left(\tfrac{1}{2}\right)(16)^3/12 = 341.\ \text{IN}^4 < 428^4$$

NO GOOD

∴ USE OPTION A, 1/2" FLANGE PL'S TOP AND BOTTOM OVER CENTER 1/3 OF SPAN OF W18x50 (A36).

USE FOR ALL MEMBERS, BOTH PANELS 1 AND 2

DEVELOP DETAILS DURING P AND S.

6/27/94

DISCHARGE BASIN STRUCTURAL DESIGN

I) GENERAL:

1. BASIC DESIGN CRITERIA:

   $f'_c = 3000$ PSI

   $f_y = 60000$ PSI

   $\rho = .25\,\rho_b$ (LIMIT)

   ASSUME SINGLY REINFORCED SECTIONS

2. IN LIEU OF A MORE ADVANCED ANALYSIS (I.E. FINITE ELEMENT MODEL OF DISCHARGE BASIN), DESIGN COMPONENTS AS FOLLOWS:

   A. SIDE WALLS AND SLAB: CHECK U-SHAPE.

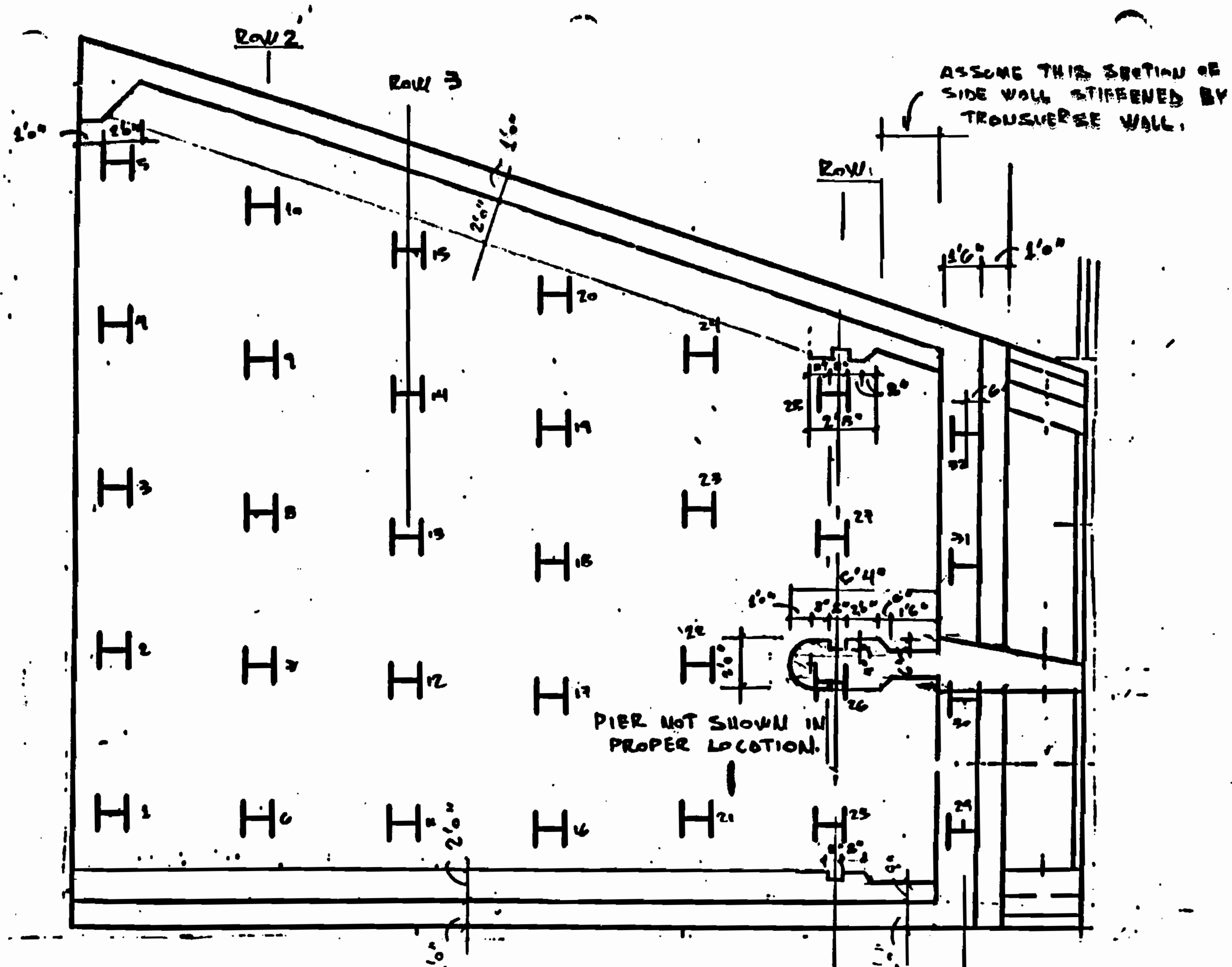
   B. PIER BETWEEN GATES AND SLAB: CHECK PIER SIZE AND EFFECT OF PIER ON SLAB

   C. BACKWALL: CHECK SIZE

3. ITEMS TO BE DETAILED DURING P&S:

   A. SIDE WALL SECTION BETWEEN BACKWALL AND STOP LOG SLOTS

   B. EXTENSIONS BETWEEN BACKWALL AND EXISTING PUMPING HOUSE WALL.



PILING PLAN FOR BASIN

II) SIDE WALLS AND SLAB: ANALYZE 1-FT STRIP OF U-SHAPE. COMPARE SLAB FORCES DUE TO PILE HEAD FIXITY ASSUMPTIONS.

1. CFRAME MODELS:

A. BASIN1A: 5-PILE SECTION, LOAD CASE 1 W/ PILE HEAD SPRING CONSTANTS.

NOTE: USE ROW 3, PILES 11-15, FROM CPGA MODEL. THE PILE REACTIONS AT THIS ROW ARE BASICLY EQUAL IN COMPRESSION. PILE REACTIONS IN ROWS 1 AND 2 ARE UNEQUAL DUE TO ECCENTRIC LOADING BECAUSE OF THE BASIN'S SHAPE AND THE RIGID PILE CAP ASSUMPTION WHICH DISTRIBUTES LOAD BASED ON POSITION. ROW 3 REACTIONS WILL BE MORE EASILY RESOLVED W/ THE CFRAME MODEL RESULTS.

B. BASIN1B: 5-PILE SECTION AS ABOVE W/ PINNED SUPPORTS AT PILE HEADS

C. BASIN2A: 4-PILE SECTION, LOAD CASE 1, W/ SPRING SUPPORTS AT PILE HEADS => CORRESPONDS TO ROW 7, PILES 29 TO 32 FROM CPGA MODEL.

D. BASIN 2B: H-SECTION AS ABOVE w/ LOAD CASE V

2. BASIN 1A:

PILE HEAD SPRING CONSTANTS: BASED ON CPGD MANUAL

AXIAL: CAE/L_E

$C = 2.0 \Rightarrow$ FRICTION PILE

$A = HP14 \times 73 \Rightarrow 21.4 \text{ IN}^2$, $E = 29000$ KSI

L_E = EMBEDDED PILE LENGTH $\Rightarrow 40' \Rightarrow 480"$

$B33 = 2(21.4)(29000)/480 = 2586$ K/IN

LATERAL: CEI/T^3 FOR LINEAR Es, $C_o = .411$

$T = [EI/nh]^{1/5}$ USE PILE CURVES

$nh = 1517./[[29'\text{-}12'](12"/')] = 7.43$ LB/IN^3

SPACING REDUCTION = .5

$nh = .5(7.43)(13.61) = 50.5$ LB/IN^3

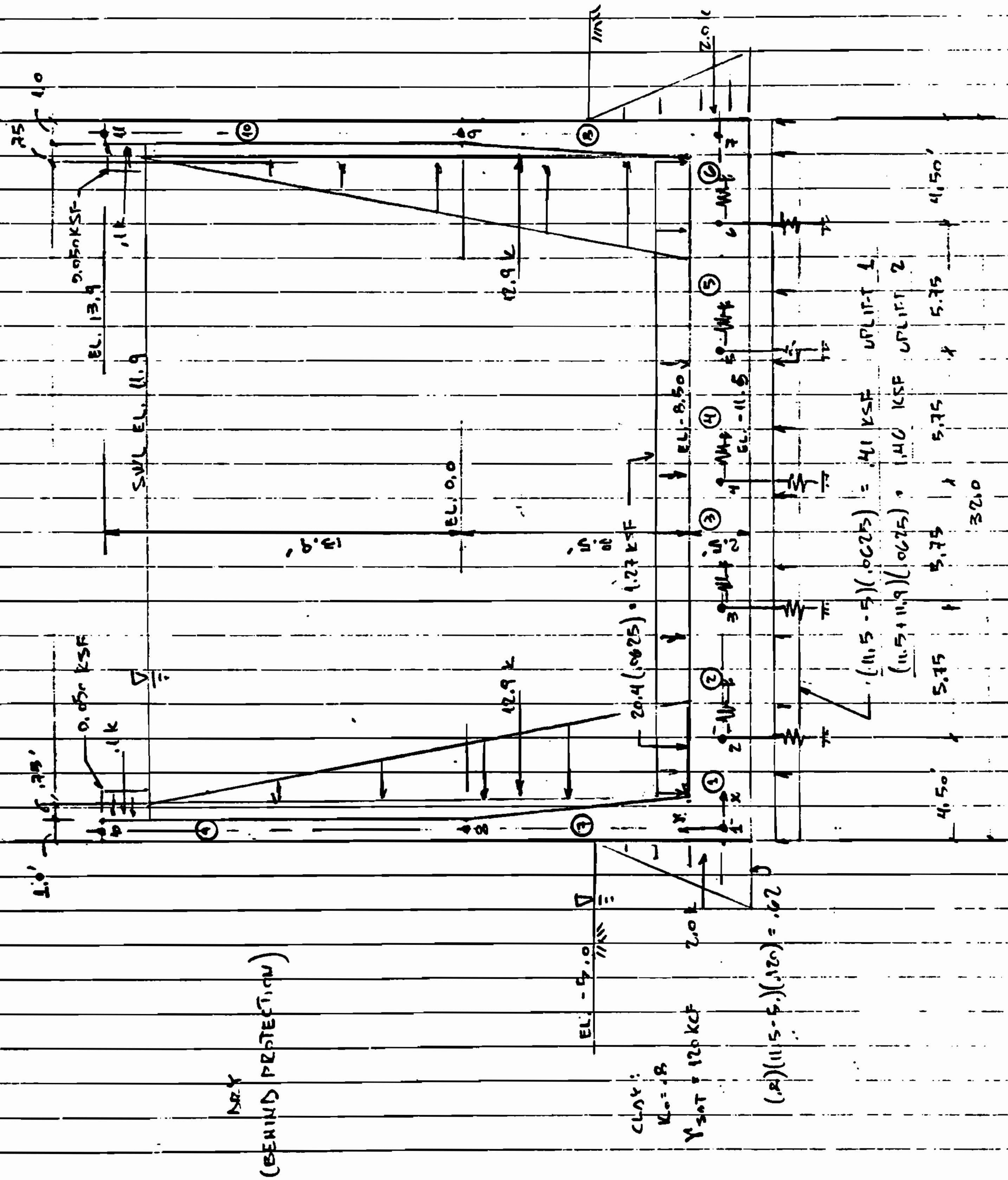
$I_{MIN} = 261.$ IN^4

$T = [29000000(261)/50.5]^{1/5} = 43.1"$

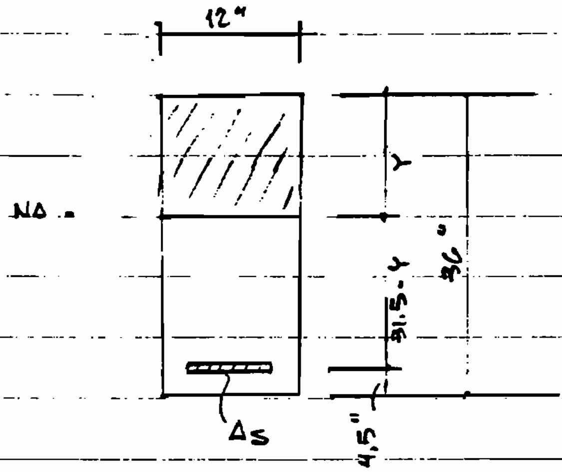
$\therefore$ $B11 = .411(29000)(261)/43.1^3 = 38.7$ K/IN

DISCHARGE BASIN: BASIN 1

5 PILE SECTION



MEMBER PROPERTIES: CHECK CRACKED TRANSFORMED SECTION



USE $A_s = 2.5 \text{ in}^2$

$n = E_s/E_c = 29{,}000{,}000 \,/\, 57000\sqrt{3000} = 9.28 \Rightarrow 9.0$

FIND N.A. : $Y(12)(Y/2) = nA_s(31.5 - Y)$

$6Y^2 = 9(2.5)(31.5 - Y)$

$= 708.75 - 22.5Y$

$6Y^2 + 22.5Y - 708.75 = 0$

$Y = 9.1"$

$I_{cr} = bY^3/3 + nA_s(31.5 - Y)^2$

$= 12(9.1)^3/3 + 9.(2.5)(31.5 - 9.1)^2$

$= 3014. + 11289.6 = 14303.6 \text{ in}^4$

$I_g = 12(36)^3/12 = 46656. \text{ in}^4$

A = AXIAL AREA $\Rightarrow$ USE $12(36) = 432. \text{ in}^2$

AS = SHEAR AREA $\Rightarrow$ USE $12(9.1) = 109.2 \text{ in}^2$

STEM BASE : THICKNESS = 36"

USE MEMBER 7, JOINT 1 FORCES

COMB 1 RESULTS

FLEXURE

1. MEMBER CAPACITY BASED ON $p = .25\, p_b$

$$p_b = \frac{.85 \beta_1 f'_c\, 87}{f_y (87 + f_y)} = \frac{.85^2 (3)(87)}{60 (87 + 60)} = 0.0214$$

$$p = .25 (.0214) = .00535$$

$$A_s = p\, b_w d = .00535 (12)(31.5) = 2.02 \text{ IN}^2$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$a = A_s f_y / .85 f'_c b = 2.02 (60) / .85 (3)(12) = 3.96"$$

$$\phi M_n = .9 (2.02)(60) \left[31.5 - 3.96/2 \right] = 3220. \text{ K-IN}$$

2. $M_u = (1.3)(1.7)(1272) = 2811. \text{ K-IN}$

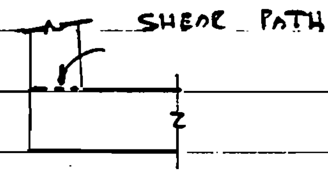
3. $\phi M_n > M_u$ OK

4. USE No. 11 @ $\left(1.56 / 2.02 \right) (12) = 9.26 \Rightarrow 9"$

SHEAR AT STEM BASE:

1. MEMBER CAPACITY

FLEXURAL SHEAR WILL OCCUR ALONG SAME PATH AS DIRECT SHEAR FOR COLD JOINT



FLEXURAL SHEAR CAPACITY = CONCRETE ONLY

$\phi V_c = \phi 2 \sqrt{f'_c}\, bd = .85(2)(3000)^{\frac{1}{2}}(12)(3.5)$

$= 35196$ LB $= 35.2$ K ⇐ GOVERNS

SHEAR FRICTION:

$\phi V_n = \phi A_{vf} f_y \mu$

A_{vf} ⇒ USE TEMP. STEEL

∴ $.0028(12)(36) = 1.21 \text{ in}^2$

μ = USE INTENTIONALLY ROUGHED SURFACE

$= 1.0\lambda = 1.(1) = 1$

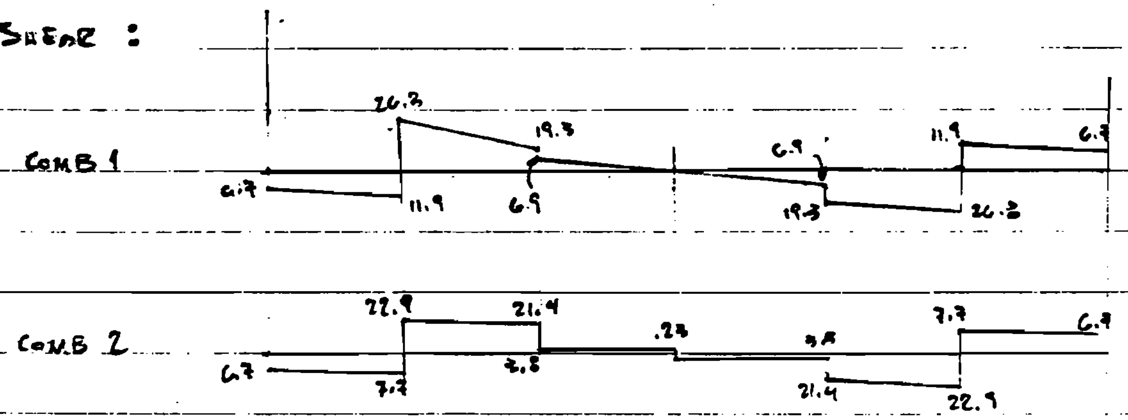
$\phi V_n = .85(1.21)(60)(1) = 61.7$ K

2. $V_u = (1.3)(1.7)(11.2) = 24.75$ K

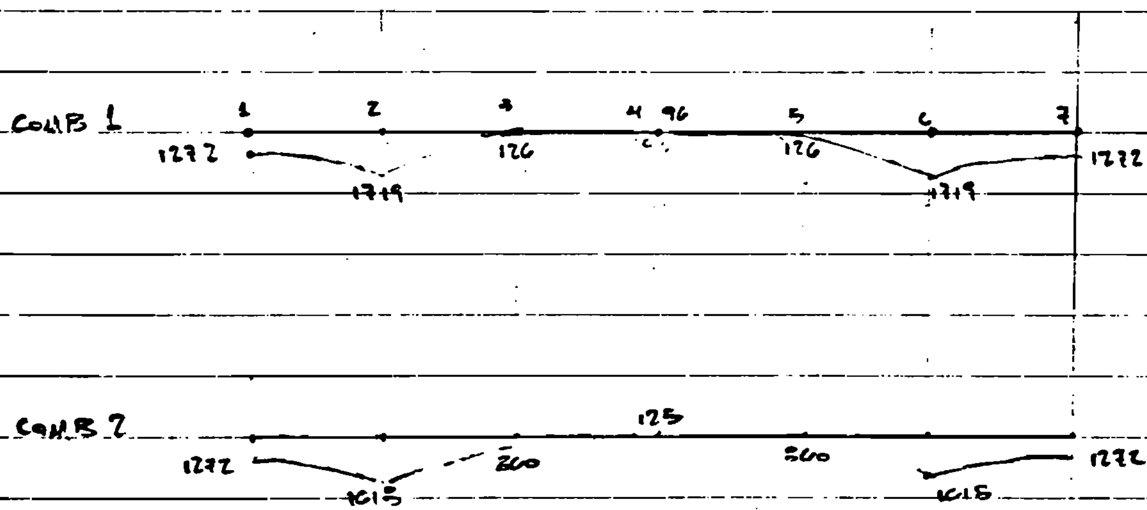
3. $\phi V_c > V_u$ OK ∴ NO SHEAR REINFORCEMENT

③ BASE SLAB : THICKNESS => 36"

SHEAR :



MOMENT :



FLEXURE AT BASE SLAB:

1. MEMBER CAPACITY BASED ON $\rho = .25 \rho_b$

$\phi M_n = 3220.$ K·IN SEE P. 7.

2. $M_u = 1.3(1.7)(1719) = 3788.$ K·IN

3. FIND REQ'D STEEL:

$$M_u = \phi M_n = \phi A_s f_y (d - a/2)$$

$$a = A_s(60) / .85(3)(12) = 1.96\ A_s$$

$$3788 = .9\ A_s(60)(31.5 - .98 A_s)$$

$$3788 = 1701\ A_s - 52.9\ A_s^2$$

$$\Rightarrow \quad 52.9\ A_s^2 - 1701\ A_s + 3788 = 0$$

$$A_s = 2.40\ \text{IN}^2$$

$$\rho = 2.40 / 12(31.5) = 0.00637$$

$$.00637 / .0214 \Rightarrow \quad \rho = .29\ \rho_b \Rightarrow \text{OK}\ ✓$$

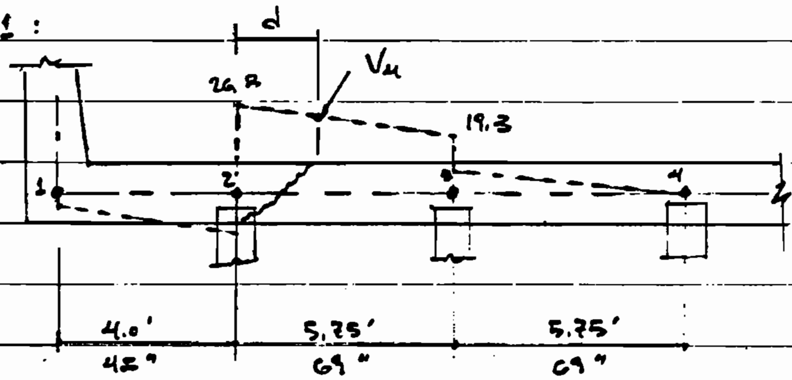
4. USE No. 11 @ $(1.56/2.40)(12) = 7.8" \Rightarrow 6"$

SHEAR AT BASE SLAB :

1. FLEXURAL SHEAR CRACKING :

CHECK IF $V_u > \phi V_c$. ACI RECOMMENDS SHEAR R/S IF $V_u > \frac{1}{2} \phi V_c$ EXCEPT FOR SLABS, WHERE LOAD SHARING IS POSSIBLE.

COMB 1 :



SHEAR: FLEXURAL SHEAR CRACKING FOR ONE-WAY SLAB

a. CHECK ϕV_c :

$$\phi V_c = \phi 2 \sqrt{f'_c}\, b_w d = (.85) 2 (3000)^{1/2} (12)(31.5) = 35196 \text{ LB.}$$

$$\phi V_c = \phi (1.9 \sqrt{f'_c} + 2500 \rho_w V_u d / M_u)\, b_w d$$

V_u @ d FROM JOINT 2:

$$V = 26.3 - ({}^{31.5}/_{69})(26.3 - 19.3) = 23.3 \text{ k}$$

$$V_u = 2.21 (23.3) = 51.5 \text{ k}$$

M_u @ d FROM JOINT 2:

$$M \simeq .5 (1719 + 127) = 923 \text{ k.in}$$

$$M_u = 2.21 (923) = 2040. \text{ k.in}$$

$$V_u d / M_u = 51.5 (31.5) / 2040 = 0.794 < 1.0 \checkmark$$

$$\rho_w \Rightarrow .25 \rho_b ; \quad \rho_b = \frac{.85 \beta_1 f'_c \; 87}{f_y (87 + f_y)} = \frac{.85^2 (3)(87)}{60 (87 + 60)}$$

$$\rho_b = 188.6 / 8820 = 0.0214$$

$$\therefore \rho_w = .25 (.0214) = .00535$$

$$\phi V_c = .85 \left[1.9 (3000)^{1/2} + 2500 (.00535)(.794)\right] b_w d$$

$$= .85 \left[(104.1 + 10.61)\, 12 (31.5) \right] = 36843. \text{ LBS}$$

$$\phi V_c = 36.8 \text{ k} < V_u$$

B. DEEP FLEXURAL MEMBERS

$$l_n/d = 5.75(12)/31.5 = 2.19 < 5 \checkmark$$

$$\phi V_c : \left(3.5 - 2.5\ M_u/V_u d\right)\left(1.9\sqrt{f'_c} + 2500\ \rho_w\ V_u d/M_u\right) b_w d$$

$$3.5 - 2.5\left(2040/51.5(31.5)\right) = 0.356 < 1. \text{ NO HELP}$$

*: CHECK SHEAR FOR DIFFERENT PILE HEAD SUPPORT CONDITIONS.

THE BASE SLAB SHEAR DIAGRAM IS SENSITIVE TO PILE HEAD SUPPORT DISPLACEMENTS. PINNED PILE HEADS WILL HAVE ZERO VERTICAL DISPLACEMENT. THIS CONDITION MAY CAUSE AN EVEN LARGER CHANGE IN SHEAR AT JOINT 2.

c. CHECK PUNCHING SHEAR :

$d/2 = 31.5/2 = 15.75''$

$b_o = 4(14 + 15.75) = 119.0''$

$\beta_c = 1.0$

$$V_c = (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d$$

BUT NOT EXCEED $4\sqrt{f'_c}\, b_o d$

$$V_c = 4(3000)^{1/2}(119)(31.5) = 821255.\ \text{LB} = 821.\ \text{k}$$

$$\phi V_c = .85(821) = 698.\ \text{k}$$

@ JOINT 2 : $V_u = 33.8(2.21) = 85.7\ \text{k}$

$\phi V_c > V_u$ OK

```
010 BASIN1A; 5 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.
020 KSI FT IN IN KIP
030 11 10 6 3000 0.15
040 1  0.5 0.00, 2 4.5  0.00, 3 10.25 0.00, 4 16.0  0.00,  5 21.75 0.00
050 6 27.5 0.00, 7 31.5 0.00
060 8  0.5 10., 9 31.5 10., 10  0.5 23.9, 11 31.5 23.9
090 FIX KX   38.7  2 3 4 5 6
100 FIX KY 2585.6  2 3 4 5 6
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6, 6 6 7
120 7  1 8, 8  7 9, 9  8 10, 10 9 11
130 14304  432  109  1 TO 10
150 LOAD CASE 1 0 2 1 0 0   CONCRETE DEAD WEIGHT
160 0. .45  4.00  .45  0.  1 6
170 0. .45  5.75  .45  0.  2 3 4 5
180 1 4.875  6.7  -90. 7 8
200 LOAD CASE 2 0 2 0 0 0  STORM WIND LOAD
205 11.9 -.05  13.9 -.05 0. 9
210 11.9  .05  13.9  .05 0. 10
215 LOAD CASE 3 0 2 0 0 0  SOIL AND WATER LATERAL PRESSURE
220 0.  .62 6.00  0    0. 7
225 0. -.62 6.00  0.   0. 8
240 LOAD CASE 4 0 6 0 0 0  INTERNAL HYDROSTATIC PRESSURE
242 0.  1.27 4.0   1.27  0. 1 6
244 0.  1.27 5.75  1.27  0. 2 3 4 5
245 1.5 -1.27 10.0 -0.74  0. 7
250 1.5  1.27 10.0  0.74  0. 8
255 0. -0.74 11.9  0.0  0. 9
260 0.  0.74 11.9  0.0  0. 10
275 LOAD CASE 5 1 0 0 0 0  EXTERNAL HYDROSTATIC UPLIFT 1
280 Y 0.41  1 TO 6
290 LOAD CASE 6 1 0 0 0 0  EXTERNAL HYDROSTATIC UPLIFT 2
300 Y 1.46  1 TO 6
310 COMB 1 1 1., 2 1., 3 1., 4 1., 5 1.  UPLIFT 1
320 COMB 2 1 1., 2 1., 3 1., 4 1., 6 1.  UPLIFT 2
```

1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
PROGRAM CFRAME V02.05 24JUL84
--*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*

RUN DATE = 95/02/01
RUN TIME = 15.18.23

BASIN1A; 5 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.

*** JOINT DATA ***

JOINT	X	Y	X	Y	R	KX	KY	KR
	--- FT ---		FIXITY			---KIP / IN---		IN-KIP/RAD
1	.50	.00						
2	4.50	.00				.387E+02	.259E+04	
3	10.25	.00				.387E+02	.259E+04	
4	16.00	.00				.387E+02	.259E+04	
5	21.75	.00				.387E+02	.259E+04	
6	27.50	.00				.387E+02	.259E+04	
7	31.50	.00						
8	.50	10.00						
9	31.50	10.00						
10	.50	23.90						
11	31.50	23.90						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
2	2	3	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
3	3	4	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
4	4	5	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
5	5	6	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
6	6	7	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
7	1	8	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
8	7	9	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
9	8	10	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
10	9	11	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04

*** LOAD CASE 1 CONCRETE DEAD WEIGHT

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.4500E+00	4.00	.4500E+00	.00
2	.00	.4500E+00	5.75	.4500E+00	.00
3	.00	.4500E+00	5.75	.4500E+00	.00
4	.00	.4500E+00	5.75	.4500E+00	.00
5	.00	.4500E+00	5.75	.4500E+00	.00
6	.00	.4500E+00	4.00	.4500E+00	.00

MEMBER	L FT	P KIP	ANGLE DEG
7	4.88	.6700E+01	-90.00
8	4.88	.6700E+01	-90.00

*** LOAD CASE 2 STORM WIND LOAD

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
9	11.90	-.5000E-01	13.90	-.5000E-01	.00
10	11.90	.5000E-01	13.90	.5000E-01	.00

*** LOAD CASE 3 SOIL AND WATER LATERAL PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
7	.00	.6200E+00	6.00	.0000E+00	.00
8	.00	-.6200E+00	6.00	.0000E+00	.00

*** LOAD CASE 4 INTERNAL HYDROSTATIC PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.1270E+01	4.00	.1270E+01	.00
2	.00	.1270E+01	5.75	.1270E+01	.00
3	.00	.1270E+01	5.75	.1270E+01	.00
4	.00	.1270E+01	5.75	.1270E+01	.00
5	.00	.1270E+01	5.75	.1270E+01	.00
6	.00	.1270E+01	4.00	.1270E+01	.00
7	1.50	-.1270E+01	10.00	-.7400E+00	.00
8	1.50	.1270E+01	10.00	.7400E+00	.00
9	.00	-.7400E+00	11.90	.0000E+00	.00
10	.00	.7400E+00	11.90	.0000E+00	.00

*** LOAD CASE 5 EXTERNAL HYDROSTATIC UPLIFT 1

MEMBER	DIRECTION	PROJECTED LOAD KIP / FT
1	Y	.4100E+00
2	Y	.4100E+00
3	Y	.4100E+00
4	Y	.4100E+00
5	Y	.4100E+00

6	Y	.100E+00

* LOAD CASE 6 EXTERNAL HYDROSTATIC UPLIFT 2

MEMBER	DIRECTION	PROJECTED LOAD KIP / FT
1	Y	.1460E+01
2	Y	.1460E+01
3	Y	.1460E+01
4	Y	.1460E+01
5	Y	.1460E+01
6	Y	.1460E+01

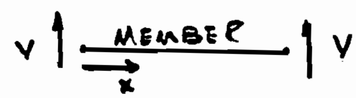
*** LOAD CASE COMBINATIONS ***

LOAD CASE	LOAD CASE FACTORS 1	2	3	4	5	6
1	1.00	1.00	1.00	1.00	1.00	.00
2	1.00	1.00	1.00	1.00	.00	1.00

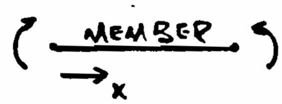
LOAD CASE 1 UPLIFT 1

SIGN CONVENTIONS:

1. SHEAR: POSITIVE



2. MOMENT: POSITIVE



JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	-.1599E-02	-.1255E+00	.3019E-02
2	-.1184E-02	-.1500E-01	.1369E-02
3	-.5913E-03	.4765E-02	-.4503E-04
4	.0000E+00	-.4157E-03	.0000E+00
5	.5924E-03	.4765E-02	.4503E-04
6	.1186E-02	-.1500E-01	-.1369E-02
7	.1600E-02	-.1255E+00	-.3019E-02
8	-.5129E+00	-.1258E+00	.4884E-02
9	.5129E+00	-.1258E+00	-.4884E-02
10	-.1356E+01	-.1258E+00	.5086E-02
11	.1356E+01	-.1258E+00	-.5086E-02

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
	2	.1119E+02	.1194E+02	-.1719E+04	-.1719E+04	48.00
2	2	.1114E+02	.2685E+02	-.1719E+04	-.1267E+03	69.00
	3	.1114E+02	-.1931E+02	-.1267E+03	-.1719E+04	.00
3	3	.1112E+02	.6995E+01	-.1267E+03	.9739E+02	63.48
	4	.1112E+02	.5375E+00	.9609E+02	-.1267E+03	.00
4	4	.1112E+02	.5375E+00	.9609E+02	.9739E+02	5.52

	5	.1112E+02	.6995E+01	-.1267E+03	-.1267E+03	69.00
5	5	.1114E+02	-.1931E+02	-.1267E+03	-.1267E+03	.00
	6	.1114E+02	.2685E+02	-.1719E+04	-.1719E+04	69.00
6	6	.1119E+02	.1194E+02	-.1719E+04	-.1272E+04	48.00
	7	.1119E+02	-.6700E+01	-.1272E+04	-.1719E+04	.00
7	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
	8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
	9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
	10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
	11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.4583E-01	.3879E+02	.0000E+00
3	.2288E-01	-.1232E+02	.0000E+00
4	-.2245E-04	.1075E+01	.0000E+00
5	-.2293E-01	-.1232E+02	.0000E+00
6	-.4588E-01	.3879E+02	.0000E+00
TOTAL	-.1125E-03	.5401E+02	

LOAD CASE 2 UPLIFT 2

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	-.1599E-02	-.1175E+00	.2923E-02
2	-.1184E-02	-.1187E-01	.1311E-02
3	-.5913E-03	.6824E-02	-.4577E-04
4	.0000E+00	.1796E-02	.0000E+00
5	.5924E-03	.6824E-02	.4577E-04
6	.1186E-02	-.1187E-01	-.1311E-02
7	.1600E-02	-.1175E+00	-.2923E-02
8	-.5013E+00	-.1178E+00	.4787E-02
9	.5014E+00	-.1178E+00	-.4788E-02
10	-.1329E+01	-.1178E+00	.4990E-02
11	.1329E+01	-.1178E+00	-.4990E-02

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
	2	.1119E+02	.7740E+01	-.1618E+04	-.1618E+04	48.00
2	2	.1114E+02	.2296E+02	-.1618E+04	-.8604E+02	69.00
	3	.1114E+02	-.2146E+02	-.8604E+02	-.1618E+04	.00
3	3	.1112E+02	.3817E+01	-.8604E+02	.1258E+03	69.00
	4	.1112E+02	-.2322E+01	.1258E+03	-.8604E+02	.00
4	4	.1112E+02	-.2322E+01	.1258E+03	.1258E+03	.00

	5	.1112E+02	.3817E+01	-.8604E+02	-.8604E+02	69.00
5	5	.1114E+02	-.2146E+02	-.8604E+02	-.8604E+02	.00
	6	.1114E+02	.2296E+02	-.1618E+04	-.1618E+04	69.00
6	6	.1119E+02	.7740E+01	-.1618E+04	-.1272E+04	48.00
	7	.1119E+02	-.6700E+01	-.1272E+04	-.1618E+04	.00
7	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
	8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
	9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
	10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
	11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.4583E-01	.3070E+02	.0000E+00
3	.2288E-01	-.1764E+02	.0000E+00
4	-.2245E-04	-.4645E+01	.0000E+00
5	-.2293E-01	-.1764E+02	.0000E+00
6	-.4588E-01	.3070E+02	.0000E+00
TOTAL	-.1125E-03	.2146E+02	

MEMBER END FORCES

MEMBER	LOAD CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
		2	.1119E+02	.1194E+02	-.1719E+04	-.1719E+04	48.00
	2	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
		2	.1119E+02	.7740E+01	-.1618E+04	-.1618E+04	48.00
2	1	2	.1114E+02	.2685E+02	-.1719E+04	-.1267E+03	69.00
		3	.1114E+02	-.1931E+02	-.1267E+03	-.1719E+04	.00
	2	2	.1114E+02	.2296E+02	-.1618E+04	-.8604E+02	69.00
		3	.1114E+02	-.2146E+02	-.8604E+02	-.1618E+04	.00
3	1	3	.1112E+02	.6995E+01	-.1267E+03	.9739E+02	63.48
		4	.1112E+02	.5375E+00	.9609E+02	-.1267E+03	.00
	2	3	.1112E+02	.3817E+01	-.8604E+02	.1258E+03	69.00
		4	.1112E+02	-.2322E+01	.1258E+03	-.8604E+02	.00
4	1	4	.1112E+02	.5375E+00	.9609E+02	.9739E+02	5.52
		5	.1112E+02	.6995E+01	-.1267E+03	-.1267E+03	69.00
	2	4	.1112E+02	-.2322E+01	.1258E+03	.1258E+03	.00
		5	.1112E+02	.3817E+01	-.8604E+02	-.8604E+02	69.00
5	1	5	.1114E+02	-.1931E+02	-.1267E+03	-.1267E+03	.00
		6	.1114E+02	.2685E+02	-.1719E+04	-.1719E+04	69.00
	2	5	.1114E+02	-.2146E+02	-.8604E+02	-.8604E+02	.00
		6	.1114E+02	.2296E+02	-.1618E+04	-.1618E+04	69.00
6	1	6	.1119E+02	.1194E+02	-.1719E+04	-.1272E+04	48.00
		7	.1119E+02	-.6700E+01	-.1272E+04	-.1719E+04	.00
	2	6	.1119E+02	.7740E+01	-.1618E+04	-.1272E+04	48.00
		7	.1119E+02	-.6700E+01	-.1272E+04	-.1618E+04	.00

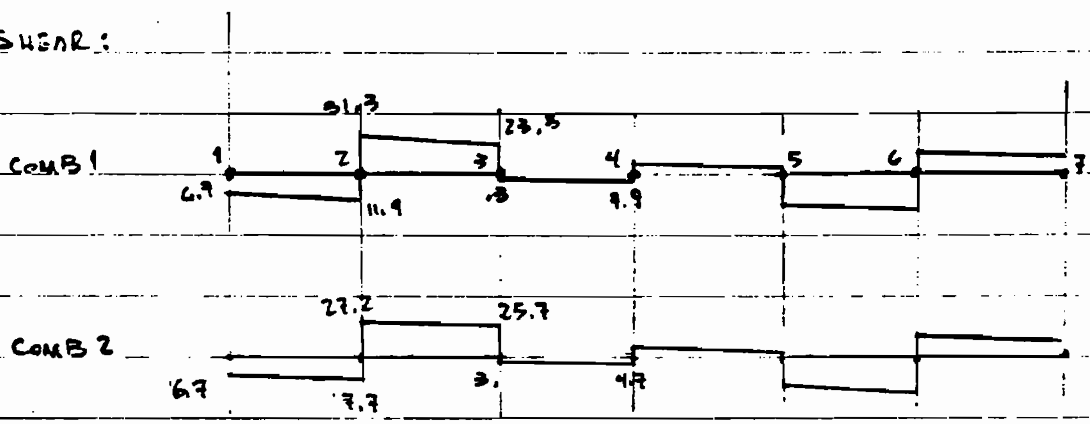
7	1	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
		8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
	2	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
		8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	1	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
		9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
	2	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
		9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	1	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
		10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
	2	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
		10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	1	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
		11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00
	2	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
		11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

2. BASIN 1B : SAME AS BASIN 1B EXCEPT PILE HEADS ARE PINNED

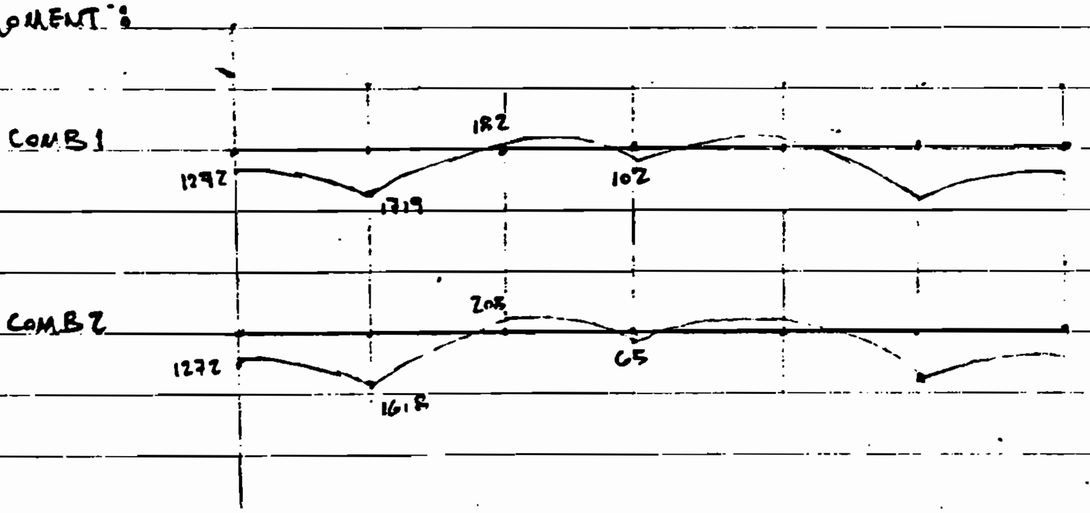
STEM BASE : SEE BASIN 1A

BASE SLAB : THICKNESS => 36"

SHEAR:



MOMENT:



AS EXPECTED, THE SHEAR AT JOINT 2 INCREASED
TO $V_u = 1.3(1.7)(31.3) = 69.2$ k

HOWEVER, THE PILE REACTION AT JOINT 1 IS 43.2 k
FOR A 5.5 FT SECTION OF BASE SLAB (THAT EACH
ROW CARRIES), THE TOTAL AT THIS PILE IS
$5.5(43.2) = 237.6$ k, THIS LOAD GROSSLY
EXCEEDS THE PILE LOAD PREDICTED BY CPGA => 56.9 k,

THE ACTUAL PILE LOAD DISTRIBUTION IS
MORE REALISTICALLY PREDICTED BY CPGA, BECAUSE
THE BASE SLAB WILL BE SUFFICIENTLY STIFF TO
CARRY THE OVERLOAD ON THE OUTER COMPRESSION
PILES, AS CALCULATED IN THE CFRAME U-SHAPE
MODEL, TO THE INNER PILES IN TWO-WAY SLAB
ACTION.

THEREFORE, CHECK THE U-SHAPE LOADS WITH
RESPECT TO THE PILE REACTIONS FROM CPGA.
DETERMINE SLAB SHEAR REINFORCEMENT BASED ON
THE RESULTING DIAGRAM.

```
010 BASIN1B; 5 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.
020 KSI FT IN IN KIP
030 11 10 6 3000 0.15
040 1  0.5 0.00, 2 4.5  0.00, 3 10.25 0.00, 4 16.0  0.00,  5 21.75 0.00
050 6 27.5 0.00, 7 31.5 0.00
060 8  0.5 10., 9 31.5 10., 10  0.5 23.9, 11 31.5 23.9
090 FIX X  2 3 4 5 6
100 FIX Y  2 3 4 5 6
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6, 6 6 7
120 7  1 8, 8  7 9, 9  8 10, 10 9 11
130 14304  432  109  1 TO 10
150 LOAD CASE 1 0 2 1 0 0   CONCRETE DEAD WEIGHT
160 0. .45  4.00  .45  0.  1 6
170 0. .45  5.75  .45  0.  2 3 4 5
180 1 4.875  6.7  -90. 7 8
200 LOAD CASE 2 0 2 0 0 0  STORM WIND LOAD
205 11.9 -.05  13.9 -.05 0. 9
210 11.9  .05  13.9  .05 0. 10
215 LOAD CASE 3 0 2 0 0 0  SOIL AND WATER LATERAL PRESSURE
220 0.  .62 6.00  0    0. 7
225 0. -.62 6.00  0.   0. 8
240 LOAD CASE 4 0 6 0 0 0  INTERNAL HYDROSTATIC PRESSURE
242 0.  1.27 4.0   1.27  0. 1 6
244 0.  1.27 5.75  1.27  0. 2 3 4 5
245 1.5 -1.27 10.0 -0.74  0. 7
250 1.5  1.27 10.0  0.74  0. 8
255 0. -0.74 11.9  0.0  0. 9
260 0.  0.74 11.9  0.0  0. 10
275 LOAD CASE 5 1 0 0 0 0  EXTERNAL HYDROSTATIC UPLIFT 1
280 Y 0.41  1 TO 6
290 LOAD CASE 6 1 0 0 0 0  EXTERNAL HYDROSTATIC UPLIFT 2
300 Y 1.46  1 TO 6
310 COMB 1 1 1., 2 1., 3 1., 4 1., 5 1.  UPLIFT 1
320 COMB 2 1 1., 2 1., 3 1., 4 1., 6 1.  UPLIFT 2
```

```
1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
 PROGRAM CFRAME V02.05 24JUL84
 *-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
```

RUN DATE = 95/02/01
RUN TIME = 17.42.01

BASINIB; 5 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.

*** JOINT DATA ***

JOINT	X	Y	X	Y	R	KX	KY	KR
	--- FT ---		-------	PIXITY	-------	---KIP / IN---		IN-KIP/RAD
1	.50	.00						
2	4.50	.00	*	*				
3	10.25	.00	*	*				
4	16.00	.00	*	*				
5	21.75	.00	*	*				
6	27.50	.00	*	*				
7	31.50	.00						
8	.50	10.00						
9	31.50	10.00						
10	.50	23.90						
11	31.50	23.90						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
2	2	3	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
3	3	4	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
4	4	5	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
5	5	6	5.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
6	6	7	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
7	1	8	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
8	7	9	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
9	8	10	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
10	9	11	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04

*** LOAD CASE 1 CONCRETE DEAD WEIGHT

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.4500E+00	4.00	.4500E+00	.00
2	.00	.4500E+00	5.75	.4500E+00	.00
3	.00	.4500E+00	5.75	.4500E+00	.00
4	.00	.4500E+00	5.75	.4500E+00	.00
5	.00	.4500E+00	5.75	.4500E+00	.00
6	.00	.4500E+00	4.00	.4500E+00	.00

MEMBER	L FT	P KIP	ANGLE DEG
7	4.88	.6700E+01	-90.00
8	4.88	.6700E+01	-90.00

*** LOAD CASE 2 STORM WIND LOAD

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
9	11.90	-.5000E-01	13.90	-.5000E-01	.00
10	11.90	.5000E-01	13.90	.5000E-01	.00

*** LOAD CASE 3 SOIL AND WATER LATERAL PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
7	.00	.6200E+00	6.00	.0000E+00	.00
8	.00	-.6200E+00	6.00	.0000E+00	.00

*** LOAD CASE 4 INTERNAL HYDROSTATIC PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.1270E+01	4.00	.1270E+01	.00
2	.00	.1270E+01	5.75	.1270E+01	.00
3	.00	.1270E+01	5.75	.1270E+01	.00
4	.00	.1270E+01	5.75	.1270E+01	.00
5	.00	.1270E+01	5.75	.1270E+01	.00
6	.00	.1270E+01	4.00	.1270E+01	.00
7	1.50	-.1270E+01	10.00	-.7400E+00	.00
8	1.50	.1270E+01	10.00	.7400E+00	.00
9	.00	-.7400E+00	11.90	.0000E+00	.00
10	.00	.7400E+00	11.90	.0000E+00	.00

*** LOAD CASE 5 EXTERNAL HYDROSTATIC UPLIFT 1

MEMBER	DIRECTION	PROJECTED LOAD KIP / FT
1	Y	.4100E+00
2	Y	.4100E+00
3	Y	.4100E+00
4	Y	.4100E+00
5	Y	.4100E+00

6	Y	.4100E+00

*** LOAD CASE 6 EXTERNAL HYDROSTATIC UPLIFT 2

MEMBER	DIRECTION	PROJECTED LOAD KIP / FT
1	Y	.1460E+01
2	Y	.1460E+01
3	Y	.1460E+01
4	Y	.1460E+01
5	Y	.1460E+01
6	Y	.1460E+01

*** LOAD CASE COMBINATIONS ***

LOAD CASE	LOAD CASE FACTORS 1	2	3	4	5	6
1	1.00	1.00	1.00	1.00	1.00	.00
2	1.00	1.00	1.00	1.00	.00	1.00

1 LOAD CASE 1 UPLIFT 1

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	-.4143E-03	-.9425E-01	.2681E-02
2	.0000E+00	.0000E+00	.1032E-02
3	.0000E+00	.0000E+00	-.1343E-03
4	.0000E+00	.0000E+00	.0000E+00
5	.0000E+00	.0000E+00	.1343E-03
6	.0000E+00	.0000E+00	-.1032E-02
7	.4143E-03	-.9425E-01	-.2681E-02
8	-.4712E+00	-.9456E-01	.4546E-02
9	.4712E+00	-.9456E-01	-.4546E-02
10	-.1258E+01	-.9456E-01	.4748E-02
11	.1258E+01	-.9456E-01	-.4748E-02

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
	2	.1119E+02	.1194E+02	-.1719E+04	-.1719E+04	48.00
2	2	.0000E+00	.3133E+02	-.1719E+04	.1825E+03	69.00
	3	.0000E+00	-.2380E+02	.1825E+03	-.1719E+04	.00
3	3	.0000E+00	-.3577E+00	.1825E+03	.1825E+03	.00
	4	.0000E+00	.7890E+01	-.1021E+03	-.1021E+03	69.00
4	4	.0000E+00	.7890E+01	-.1021E+03	.1825E+03	69.00

	5	.0000E+00	-.3577E+00	.1825E+03	-.1021E+03	.00
5	5	.0000E+00	-.2380E+02	.1825E+03	.1825E+03	.00
	6	.0000E+00	.3133E+02	-.1719E+04	-.1719E+04	69.00
6	6	.1119E+02	.1194E+02	-.1719E+04	-.1272E+04	48.00
	7	.1119E+02	-.6700E+01	-.1272E+04	-.1719E+04	.00
7	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
	8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
	9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
	10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
	11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.1119E+02	.4327E+02	.0000E+00
3	.0000E+00	-.2415E+02	.0000E+00
4	.0000E+00	.1578E+02	.0000E+00
5	.0000E+00	-.2415E+02	.0000E+00
6	-.1119E+02	.4327E+02	.0000E+00
TOTAL	-.1125E-03	.5401E+02	

1 LOAD CASE 2 UPLIFT 2

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	-.4143E-03	-.9024E-01	.2603E-02
2	.0000E+00	.0000E+00	.9910E-03
3	.0000E+00	.0000E+00	-.1290E-03
4	.0000E+00	.0000E+00	.0000E+00
5	.0000E+00	.0000E+00	.1290E-03
6	.0000E+00	.0000E+00	-.9910E-03
7	.4143E-03	-.9024E-01	-.2603E-02
8	-.4618E+00	-.9054E-01	.4468E-02
9	.4618E+00	-.9054E-01	-.4468E-02
10	-.1236E+01	-.9054E-01	.4670E-02
11	.1236E+01	-.9054E-01	-.4670E-02

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
	2	.1119E+02	.7740E+01	-.1618E+04	-.1618E+04	48.00
2	2	.0000E+00	.2722E+02	-.1618E+04	.2083E+03	69.00
	3	.0000E+00	-.2573E+02	.2083E+03	-.1618E+04	.00
3	3	.0000E+00	-.3214E+01	.2083E+03	.2083E+03	.00
	4	.0000E+00	.4709E+01	-.6505E+02	-.6505E+02	69.00
4	4	.0000E+00	.4709E+01	-.6505E+02	.2083E+03	69.00

	5	.0000E+00	-.3214E+01	.2083E+03	-.6505E+02	.00
5	5	.0000E+00	-.2573E+02	.2083E+03	.2083E+03	.00
	6	.0000E+00	.2722E+02	-.1618E+04	-.1618E+04	69.00
6	6	.1119E+02	.7740E+01	-.1618E+04	-.1272E+04	48.00
	7	.1119E+02	-.6700E+01	-.1272E+04	-.1618E+04	.00
7	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
	8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
	9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
	10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
	11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	.1119E+02	.3496E+02	.0000E+00
3	.0000E+00	-.2894E+02	.0000E+00
4	.0000E+00	.9418E+01	.0000E+00
5	.0000E+00	-.2894E+02	.0000E+00
6	-.1119E+02	.3496E+02	.0000E+00
-------------	-------------	-------------	
TOTAL	-.1125E-03	.2146E+02	

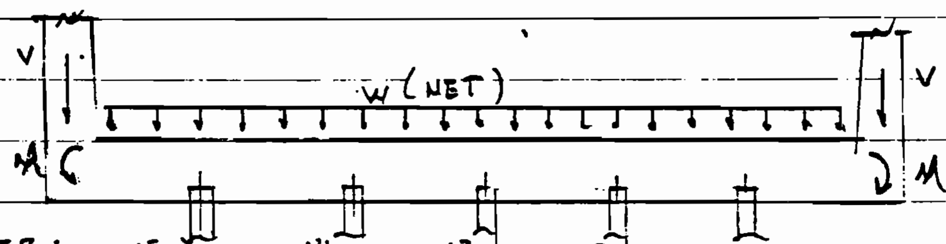
1 MEMBER END FORCES

MEMBER	LOAD CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
		2	.1119E+02	.1194E+02	-.1719E+04	-.1719E+04	48.00
	2	1	.1119E+02	-.6700E+01	-.1272E+04	-.1272E+04	.00
		2	.1119E+02	.7740E+01	-.1618E+04	-.1618E+04	48.00
2	1	2	.0000E+00	.3133E+02	-.1719E+04	.1825E+03	69.00
		3	.0000E+00	-.2380E+02	.1825E+03	-.1719E+04	.00
	2	2	.0000E+00	.2722E+02	-.1618E+04	.2083E+03	69.00
		3	.0000E+00	-.2573E+02	.2083E+03	-.1618E+04	.00
3	1	3	.0000E+00	-.3577E+00	.1825E+03	.1825E+03	.00
		4	.0000E+00	.7890E+01	-.1021E+03	-.1021E+03	69.00
	2	3	.0000E+00	-.3214E+01	.2083E+03	.2083E+03	.00
		4	.0000E+00	.4709E+01	-.6505E+02	-.6505E+02	69.00
4	1	4	.0000E+00	.7890E+01	-.1021E+03	.1825E+03	69.00
		5	.0000E+00	-.3577E+00	.1825E+03	-.1021E+03	.00
	2	4	.0000E+00	.4709E+01	-.6505E+02	.2083E+03	69.00
		5	.0000E+00	-.3214E+01	.2083E+03	-.6505E+02	.00
5	1	5	.0000E+00	-.2380E+02	.1825E+03	.1825E+03	.00
		6	.0000E+00	.3133E+02	-.1719E+04	-.1719E+04	69.00
	2	5	.0000E+00	-.2573E+02	.2083E+03	.2083E+03	.00
		6	.0000E+00	.2722E+02	-.1618E+04	-.1618E+04	69.00
6	1	6	.1119E+02	.1194E+02	-.1719E+04	-.1272E+04	48.00
		7	.1119E+02	-.6700E+01	-.1272E+04	-.1719E+04	.00
	2	6	.1119E+02	.7740E+01	-.1618E+04	-.1272E+04	48.00
		7	.1119E+02	-.6700E+01	-.1272E+04	-.1618E+04	.00

7	1	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
		8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
	2	1	-.6700E+01	-.1119E+02	.1272E+04	.1272E+04	.00
		8	.0000E+00	.4503E+01	.2251E+03	.2251E+03	120.00
8	1	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
		9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
	2	7	-.6700E+01	.1119E+02	-.1272E+04	-.2251E+03	120.00
		9	.0000E+00	-.4503E+01	-.2251E+03	-.1272E+04	.00
9	1	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
		10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
	2	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
		10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	1	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
		11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00
	2	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
		11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00

3. U-SHAPE LOADS w/ PILE REACTIONS FROM CPGA.

LOAD CASE 1 ; 1-FT STRIP



PILE NUMBER : 15 14 13 12 11

PILE REACTION
PER FT WIDTH 10.3 k 10.3 k 10.3 k 10.3 k 10.3 k

$M = 12.9(8.3) + .1(22.9) - 2.1(.67) = 108.$ K·FT

V = STEM WT = $\left[1(22.4) + .5(2)(22.4)\right](.150) = 6.7$ K/FT

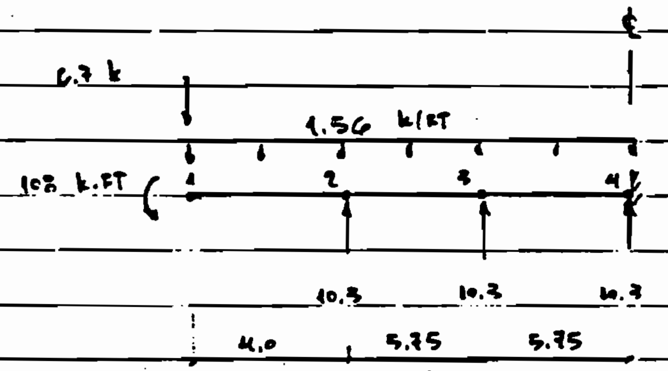
w(NET) = STILL WATER : 20.4(.0625) = 1.275 KSF
CONCRETE : 3(0.15) = 0.450
UPLIFT : 6.5(0.0625) = −0.420
1.305 KSF

HALF OF STRUCTURE:

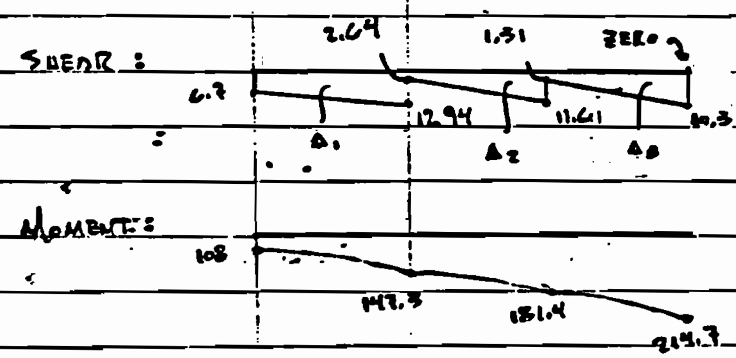
a) $\Sigma F_V = 6.7 + 1.31(15.5) - 10.3(3) = -3.9\ k$

∴ ADD 3.9 k OF VERTICAL LOAD. SPREAD ADDED LOAD OVER THE SPAN TO LIMIT ERROR IN THE PILE REACTIONS, AND SHEAR AND MOMENT DIAGRAMS. ⇒ $3.9/15.5 = .25\ k/FT$

$1.31 + .25 = 1.56\ k/FT$



$A_1 = 6.7(4.) + .5(12.94 - 6.7)(4.)$
$= 26.8 + 12.5 = 39.3\ k.FT$



$A_2 = 2.6(5.75) + .5(11.61 - 2.6)(5.75)$
$= 8.3 + 25.8 = 34.1\ k.FT$

$A_3 = 1.3(5.75) + .5(10.3 - 1.3)(5.75)$
$= 7.5 + 25.9 = 33.3\ k.FT$

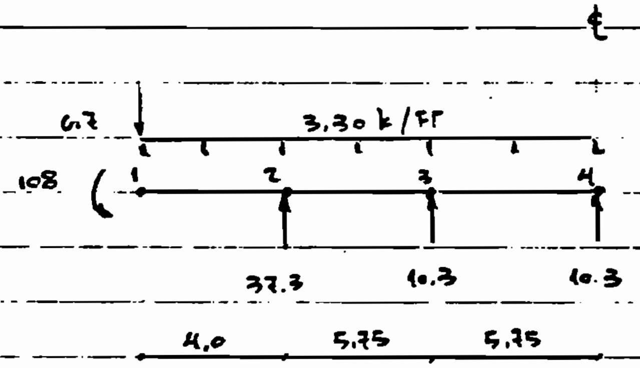
214.7 k.FT @ CENTER SPAN IS UNREALISTIC.
PILE REACTIONS NEED TO BE ADJUSTED DUE TO CONCENTRATED MOMENT.

B) INCREASE REACTION AT JOINT 2 : 108 k.FT / 4 FT = 27. k

INCREASE DISTRIBUTED LOAD BY ADDITIONAL

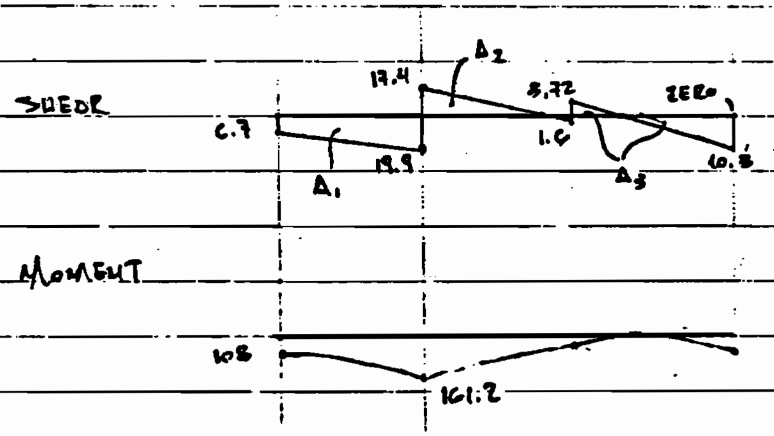
27. / 15.5 = 1.74 k / FT

1.56 + 1.74 => 3.30 k / FT



$A_1 = 6.7(4) + .5(19.9 - 6.7)(4)$

$= 26.8 + 26.4 = 53.2$ k.FT



$A_2 \approx .5(17.4)(5.75) = -50.0$ k.FT

FLEXURE :

1. FIND REQ'D STEEL

$M_u = 1.3(1.7)(161.2) = 356.2$ K.FT

$M_u = \phi A_s f_y (d - a/2)$ SEE P. 16

$356.2(12) = 1701 A_s - 52.9 A_s^2$

$52.9 A_s^2 - 1701 A_s + 4272. = 0$

$A_s = 2.74$ IN^2

$\rho = 2.74 / 12(31.5) = .00726$

$.00726 / .0214 \Rightarrow \rho = .33 \rho_b$

2. INCREASE $f'_c = 4000$ PSI AND FIND REQ'D STEEL

$a = A_s(60)/.85(4)(12) = 1.47 A_s$

$39.7 A_s^2 - 1701 A_s + 4272 = 0$

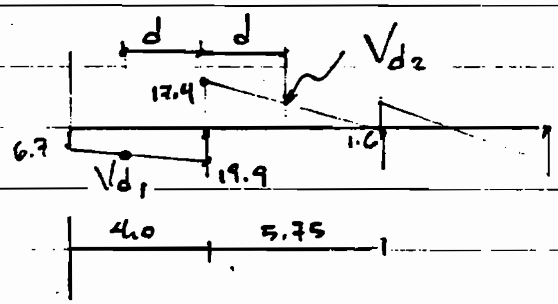
$A_s = 2.07$ IN^2, $\rho = 2.07 / 12(31.5) = .00709$

$.00709 / .0214 \Rightarrow \rho = .33 \rho_b$ ∴ STICK W/ $f'_c = 3000 \Rightarrow$ OK

SHEAR :

1. FLEXURAL SHEAR :

FROM P. 12 , $\phi V_c = 35.1$ k



$V_{d1} > V_{d2}$

$V_{d_1} = 19.9 - (^{31.5}/_{48})(19.9 - 6.7) = 11.2$ k

$V_u = 1.3(1.7)(11.2) = 24.8$ k

$\phi V_c > V_u$ NO SHEAR REINFORCEMENT REQ'D.

B. BASIN 2 : 4-PILE SECTION @ GATES w/
PILE SPRINGS => CORRESPONDS TO
ROW 7, PILES 29 TO 32 IN CPGA
MODEL.

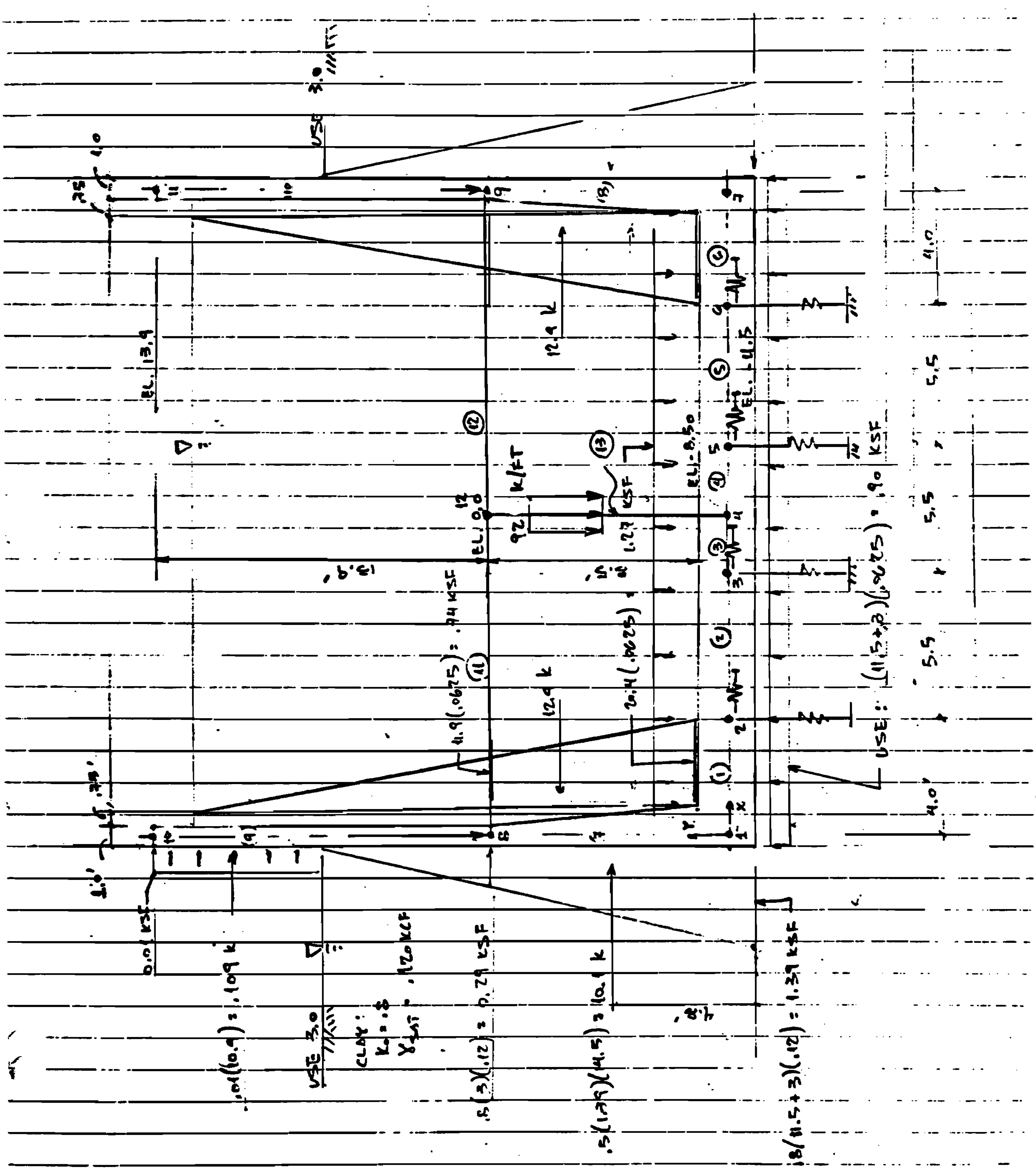
GENERAL :

1. INCLUDE LONGITUDINAL WALL AND PIER IN
MODEL. MAKE MEMBERS RIGID, I.E.
I = 99999, A = AS = 999.

UPLIFT 1

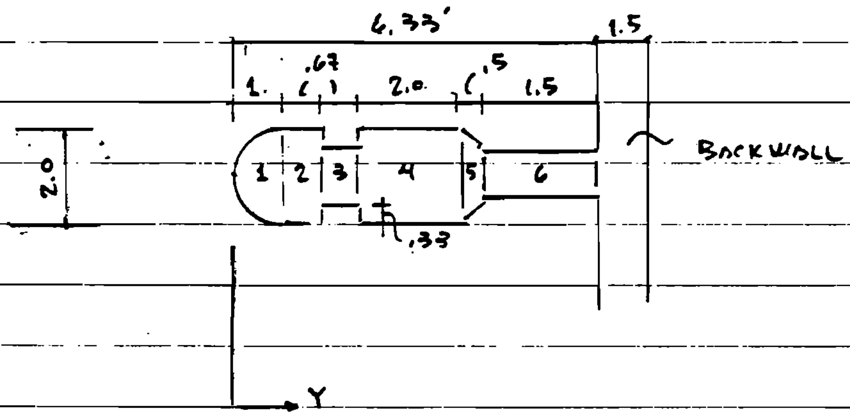
DISCHARGE BASIN: BASIN 2A

4. PILE SECTION



FIND DEAD LOADS :

A. CENTER PIER :



$A = .5(\pi)(1)^2 + 2(.67) + 1.33(.67) + 2(2.) + .5(1.5) + 1.5(1.)$

$= 1.57 + 1.34 \quad 0.89 \quad 4.0 \quad 0.75 \quad 1.5$

$= 10.05 \Rightarrow 10.0 \text{ FT}^2$

$\bar{y} = [.75(1.57) + 1.33(1.34) + 2(.89) + 3.34(4.) + 4.59(.75) + 5.6(1.5)] / 10.$

$= [1.18 + 1.78 + 1.78 + 13.36 + 3.44 + 8.38] / 10 = 3.0'$

WEIGHT $= (22.4)(10.)(.15) = 33.6$ k

B. BACKWALL :

WALL OVER GATE OPENINGS :

$(22.4)(1.5)(.15) = 3.5$ k / FT (IGNORES MID-SECTION WHERE WALL IS FULL HEIGHT)

WALL OVER GATE OPENINGS:

$$22.5 - \left[\overset{5.5}{1 + 2.(2.25)}\right] = 17.0'$$

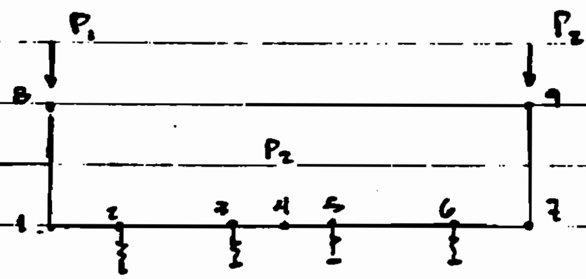
$$\text{WEIGHT} = 17.\ (3.5) = 59.5\ k$$

FULL HEIGHT SECTIONS:

$$(22.4)(5.5)(1.5)(.15) = 27.7\ k$$

$$\text{TOTAL WEIGHT} = 59.5 + 27.7 = 87.2\ k$$

DISTRIBUTE AS FOLLOWS:



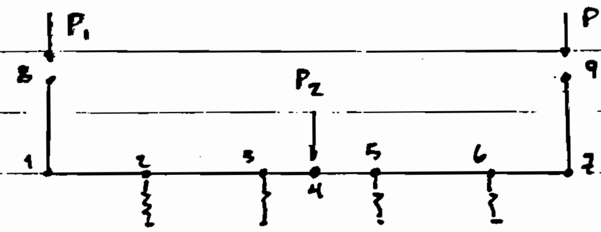
$$P_1 = 2.25/5.5\ (27.7) + .25(59.5)$$

$$= 11.3 + 14.9 = 26.2\ k\ /4. = [6.5\ k\ \text{FOR } 1'\ \text{STRIP}]$$

$$P_2 = 1/5.5\ (27.7) + .5\ (59.5) =$$

$$= 5.0 + 29.75 = 34.7\ k$$ => SEE PIER FOOTPRINT LOADING

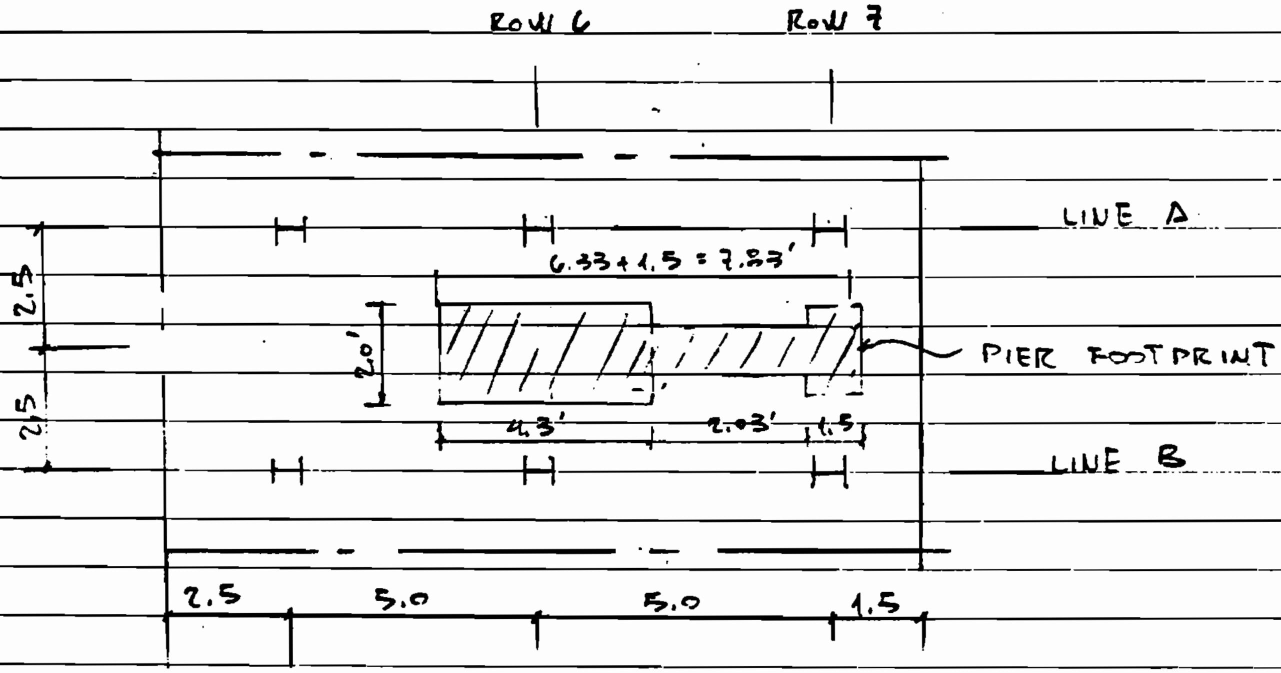
C. GATES: USE 20 k EACH => 40 k TOTAL



$P_1 = .25\,(40) = 10.\ k\ /\ 4. = \left[2.5\ k\ \text{FOR } 1'\ \text{STRIP}\right]$

$P_2 = .50\,(40) = 20.\ k$ SEE PIER FOOTPRINT LOADING

FIND PIER FOOTPRINT LOADING ON SLAB :



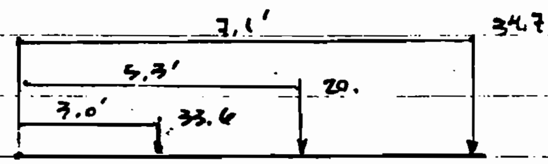
PARTIAL PLAN OF SLAB @ PIER

- PILE LOCATIONS ARE NOT ACTUAL ⇒ SEE PILE LAYOUT
- @ ROW 7, THE PIER IS APPROX. CENTERED BETWEEN LINE A AND B

* ANALYZE ONE (1) FT STRIP, TRANSVERSE SECTION OF BASIN @ ROW 7.

FOR SIMPLICITY, USE $S = bd^2/6 = 2.(7.83)^2/6 = 20.4\ FT^3$

DEAD LOADS:



$$\bar{Y} = 33.6(3) + 20.(5.3) + 34.7(7.1) \,/\, (33.6 + 20 + 34.7) =$$
$$= 453.1 \,/\, 88.3 = 5.1'$$

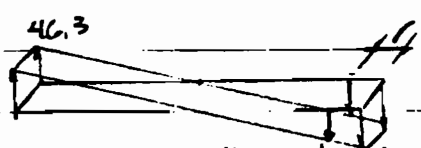
$$M = 88.3\left(5.1 - 7.83/2\right) = 104.6 \text{ k·FT}$$

LIVE LOADS FOR CASE 1:

$$M = 12.9 \text{ k/FT}\ (6.8)\ \big[\,\underbrace{.5(17) + 1}_{\text{WALL LENGTH}}\,\big] = 840. \text{ k·FT}$$

6. FOOTPRINT LOADING

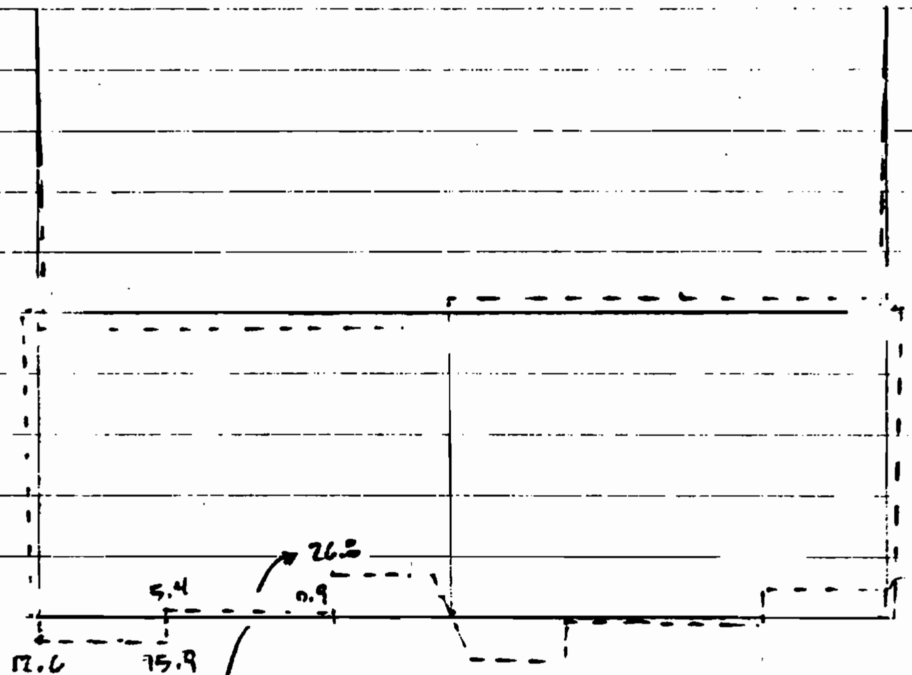
$$f = M_T/S = [105 + 840]/20.4 = 46.3 \text{ k/FT}$$



AVG OVER 1' FT STRIP: $[46.3 + 40.4]/2 = 43.3$ k/FT

SHEAR:

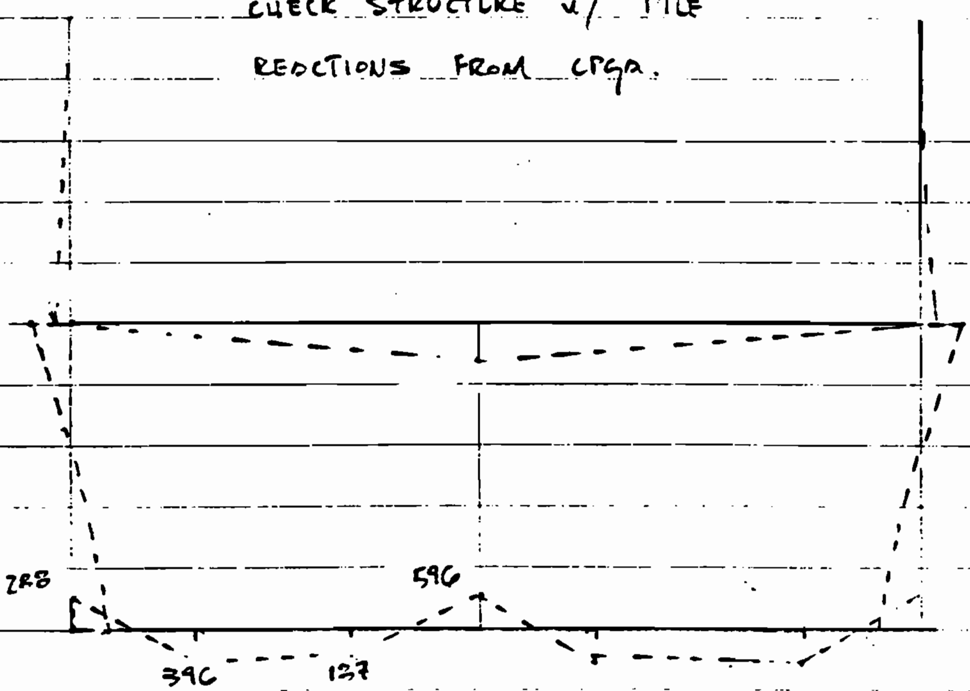
COMB 1



HIGH SHEAR LOAD IS UNREALISTIC.
CHECK STRUCTURE W/ PILE
REACTIONS FROM CPGA.

MOMENT:

COMB 1



```
010 BASIN2A; 4 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.
020 KSI FT IN IN KIP
030 12 13 6 3000 0.15
040 1  0.5 0., 2 4.5  0., 3 10.0 0., 4 12.75 0. 5 15.5 0., 6 21.0 0.
050 7 25.0  0.
060 8  0.5 10., 9 25.0 10., 10  0.5 23.9, 11 25. 23.9
070 12 12.75 10.
090 FIX KX 38.7     2 3 5 6
100 FIX KY 2585.6  2 3 5 6
110 1  1 2, 2  2 3, 3  3 4, 4  4 5, 5 5 6, 6 6 7
120 7  1 8, 8  7 9, 9  8 10, 10 9 11, 11 8 12, 12 12 9, 13 4 12
130 14304  432  109  1 TO 10
135 99999  9999 9999 11 TO 13
145 LOAD CASE 1 0 3 1 0 0  DEAD WEIGHT EXCEPT PIER LOAD
150 0. .45  4.00  .45  0.  1 6
155 0. .45  5.50  .45  0.  2 5
160 0. .45  2.75  .45  0.  3 4
165 1 4.875  15.7  -90. 7 8
170 LOAD CASE 2 0 2 0 0 0  PIER LOAD
175 2.25  43  2.75  43  0.  3
180 0.    43  0.50  43  0.  4
200 LOAD CASE 3 0 2 0 0 0  STORM WIND LOAD
205 11.9 -.05  13.9 -.05 0. 9
210 11.9  .05  13.9  .05 0. 10
215 LOAD CASE 4 0 2 0 0 0  SOIL AND WATER LATERAL PRESSURE
220 0.  .62 6.00  0    0. 7
225 0. -.62 6.00  0.   0. 8
240 LOAD CASE 5 0 6 0 0 0  INTERNAL HYDROSTATIC PRESSURE
242 0.  1.27 4.0   1.27  0. 1 6
244 0.  1.27 5.75  1.27  0. 2 3 4 5
245 1.5 -1.27 10.0 -0.74  0. 7
250 1.5  1.27 10.0  0.74  0. 8
255 0. -0.74 11.9  0.0  0. 9
260 0.  0.74 11.9  0.0  0. 10
275 LOAD CASE 6 1 0 0 0 0  EXTERNAL HYDROSTATIC UPLIFT 1
280 Y 0.90  1 TO 6
310 COMB 1 1 1., 2 1., 3 1., 4 1., 5 1., 6 1.  UPLIFT 1
```

1*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
PROGRAM CFRAME V02.05 24JUL84
--*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*

RUN DATE = 95/02/02
RUN TIME = 17.18.35

TA
[illegible]; 4 PILE SECTION, 1 FT STRIP, CASE I, CRACKED MEMBER PROP.

*** JOINT DATA ***

JOINT	X	Y	X	Y	R	KX	KY	KR
	--- FT ---			---FIXITY---		---KIP / IN---		IN-KIP/RAD
1	.50	.00						
2	4.50	.00				.387E+02	.259E+04	
3	10.00	.00				.387E+02	.259E+04	
4	12.75	.00						
5	15.50	.00				.387E+02	.259E+04	
6	21.00	.00				.387E+02	.259E+04	
7	25.00	.00						
8	.50	10.00						
9	25.00	10.00						
10	.50	23.90						
11	25.00	23.90						
12	12.75	10.00						

*** MEMBER DATA ***

MEMBER	END A	END B	LENGTH FT	I IN**4	A IN**2	AS IN**2	E KSI	G KSI
1	1	2	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
2	2	3	5.50	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
3	3	4	2.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
4	4	5	2.75	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
5	5	6	5.50	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
6	6	7	4.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
7	1	8	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
8	7	9	10.00	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
9	8	10	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
10	9	11	13.90	.1430E+05	.4320E+03	.1090E+03	.3000E+04	.1304E+04
11	8	12	12.25	.1000E+06	.9999E+04	.9999E+04	.3000E+04	.1304E+04
12	12	9	12.25	.1000E+06	.9999E+04	.9999E+04	.3000E+04	.1304E+04
13	4	12	10.00	.1000E+06	.9999E+04	.9999E+04	.3000E+04	.1304E+04

*** LOAD CASE 1 DEAD WEIGHT EXCEPT PIER LOAD

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.4500E+00	4.00	.4500E+00	.00
2	.00	.4500E+00	5.50	.4500E+00	.00
3	.00	.4500E+00	2.75	.4500E+00	.00

4	.00	.4500E+00	2.75	.4500E+00	.00
5	.00	.4500E+00	5.50	.4500E+00	.00
6	.00	.4500E+00	4.00	.4500E+00	.00

MEMBER	L FT	P KIP	ANGLE DEG
7	4.88	.1570E+02	-90.00
8	4.88	.1570E+02	-90.00

*** LOAD CASE 2 PIER LOAD

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
3	2.25	.4300E+02	2.75	.4300E+02	.00
4	.00	.4300E+02	.50	.4300E+02	.00

*** LOAD CASE 3 STORM WIND LOAD

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
9	11.90	-.5000E-01	13.90	-.5000E-01	.00
10	11.90	.5000E-01	13.90	.5000E-01	.00

*** LOAD CASE 4 SOIL AND WATER LATERAL PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
7	.00	.6200E+00	6.00	.0000E+00	.00
8	.00	-.6200E+00	6.00	.0000E+00	.00

*** LOAD CASE 5 INTERNAL HYDROSTATIC PRESSURE

MEMBER	LA FT	PA KIP / FT	LB FT	PB KIP / FT	ANGLE DEG
1	.00	.1270E+01	4.00	.1270E+01	.00
2	.00	.1270E+01	5.50	.1270E+01	.00
3	.00	.1270E+01	2.75	.1270E+01	.00
4	.00	.1270E+01	2.75	.1270E+01	.00
5	.00	.1270E+01	5.50	.1270E+01	.00
6	.00	.1270E+01	4.00	.1270E+01	.00
7	1.50	-.1270E+01	10.00	-.7400E+00	.00
8	1.50	.1270E+01	10.00	.7400E+00	.00
9	.00	-.7400E+00	11.90	.0000E+00	.00
10	.00	.7400E+00	11.90	.0000E+00	.00

*** LOAD CASE 6 EXTERNAL HYDROSTATIC UPLIFT 1

MEMBER	DIRECTION	PROJECTED LOAD KIP / FT
1	Y	.9000E+00
2	Y	.9000E+00
3	Y	.9000E+00
4	Y	.9000E+00
5	Y	.9000E+00
6	Y	.9000E+00

*** LOAD CASE COMBINATIONS ***

LOAD CASE	LOAD CASE FACTORS 1	2	3	4	5	6
1	1.00	1.00	1.00	1.00	1.00	1.00

1 LOAD CASE 1 UPLIFT 1

JOINT DISPLACEMENTS

JOINT	DX IN	DY IN	DR RAD
1	.2840E-03	-.2816E-01	.2736E-03
2	.1916E-03	-.8248E-02	.2277E-03
3	.6497E-04	-.1002E-01	-.1835E-03
4	.0000E+00	-.2062E-01	.0000E+00
5	-.6157E-04	-.1002E-01	.1835E-03
6	-.1882E-03	-.8248E-02	-.2277E-03
7	-.2806E-03	-.2816E-01	-.2736E-03
8	-.6506E-04	-.2859E-01	.7075E-04
9	.6902E-04	-.2859E-01	-.7076E-04
10	-.4055E-01	-.2859E-01	.2731E-03
11	.4055E-01	-.2859E-01	-.2731E-03
12	.0000E+00	-.2064E-01	.0000E+00

MEMBER END FORCES

MEMBER	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	-.2495E+01	-.1263E+02	.2883E+03	.2883E+03	.00
	2	-.2495E+01	.1591E+02	-.3966E+03	-.3966E+03	48.00
2	2	-.2487E+01	.5419E+01	-.3966E+03	-.1877E+03	66.00
	3	-.2487E+01	-.9091E+00	-.1877E+03	-.3966E+03	.00
3	3	-.2485E+01	.2683E+02	-.1877E+03	.5958E+03	33.00
	4	-.2485E+01	-.3072E+01	.5958E+03	-.1877E+03	.00
4	4	-.2485E+01	-.3072E+01	.5959E+03	.5959E+03	.00
	5	-.2485E+01	.2683E+02	-.1877E+03	-.1877E+03	33.00

5	5	-.2487E+01	-.9092E+00	-.1877E+03	-.1877E+03	.00
	6	-.2487E+01	.5419E+01	-.3966E+03	-.3966E+03	66.00
6	6	-.2495E+01	.1591E+02	-.3966E+03	.2883E+03	48.00
	7	-.2495E+01	-.1263E+02	.2883E+03	-.3966E+03	.00
7	1	-.1263E+02	.2495E+01	-.2883E+03	.3065E+03	120.00
	8	.3072E+01	-.9177E+01	.3065E+03	-.2883E+03	.00
8	7	-.1263E+02	-.2495E+01	.2883E+03	.2883E+03	.00
	9	.3072E+01	.9177E+01	-.3065E+03	-.3065E+03	120.00
9	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00
	10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
	11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00
11	8	.1368E+02	-.3072E+01	.8141E+02	.8141E+02	.00
	12	.1368E+02	.3072E+01	-.3702E+03	-.3702E+03	147.00
12	12	.1368E+02	.3072E+01	-.3702E+03	.8141E+02	147.00
	9	.1368E+02	-.3072E+01	.8141E+02	-.3702E+03	.00
13	4	-.6144E+01	.1171E-03	-.6676E-02	.7376E-02	120.00
	12	-.6144E+01	-.1171E-03	.7376E-02	-.6676E-02	.00

STRUCTURE REACTIONS

JOINT	FORCE X KIP	FORCE Y KIP	MOMENT IN-KIP
2	-.7416E-02	.2133E+02	.0000E+00
3	-.2514E-02	.2592E+02	.0000E+00
5	.2383E-02	.2592E+02	.0000E+00
6	.7285E-02	.2133E+02	.0000E+00
TOTAL	-.2636E-03	.9449E+02	

1 MEMBER END FORCES

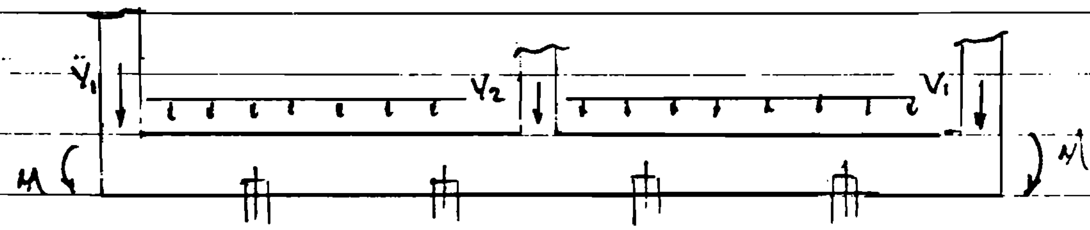
MEMBER	LOAD CASE	JOINT	AXIAL KIP	SHEAR KIP	MOMENT IN-KIP	MOMENT EXTREMA IN-KIP	LOCATION IN
1	1	1	-.2495E+01	-.1263E+02	.2883E+03	.2883E+03	.00
		2	-.2495E+01	.1591E+02	-.3966E+03	-.3966E+03	48.00
2	1	2	-.2487E+01	.5419E+01	-.3966E+03	-.1877E+03	66.00
		3	-.2487E+01	-.9091E+00	-.1877E+03	-.3966E+03	.00
3	1	3	-.2485E+01	.2683E+02	-.1877E+03	.5958E+03	33.00
		4	-.2485E+01	-.3072E+01	.5958E+03	-.1877E+03	.00
4	1	4	-.2485E+01	-.3072E+01	.5959E+03	.5959E+03	.00
		5	-.2485E+01	.2683E+02	-.1877E+03	-.1877E+03	33.00
5	1	5	-.2487E+01	-.9092E+00	-.1877E+03	-.1877E+03	.00
		6	-.2487E+01	.5419E+01	-.3966E+03	-.3966E+03	66.00
6	1	6	-.2495E+01	.1591E+02	-.3966E+03	.2883E+03	48.00
		7	-.2495E+01	-.1263E+02	.2883E+03	-.3966E+03	.00
7	1	1	-.1263E+02	.2495E+01	-.2883E+03	.3065E+03	120.00
		8	.3072E+01	-.9177E+01	.3065E+03	-.2883E+03	.00
8	1	7	-.1263E+02	-.2495E+01	.2883E+03	.2883E+03	.00
		9	.3072E+01	.9177E+01	-.3065E+03	-.3065E+03	120.00
9	1	8	.0000E+00	-.4503E+01	.2251E+03	.2251E+03	.00

		10	.0000E+00	.0000E+00	.0000E+00	.0000E+00	166.80
10	1	9	.0000E+00	.4503E+01	-.2251E+03	.0000E+00	166.80
		11	.0000E+00	.0000E+00	.0000E+00	-.2251E+03	.00
11	1	8	.1368E+02	-.3072E+01	.8141E+02	.8141E+02	.00
		12	.1368E+02	.3072E+01	-.3702E+03	-.3702E+03	147.00
12	1	12	.1368E+02	.3072E+01	-.3702E+03	.8141E+02	147.00
		9	.1368E+02	-.3072E+01	.8141E+02	-.3702E+03	.00
13	1	4	-.6144E+01	.1171E-03	-.6676E-02	.7376E-02	120.00
		12	-.6144E+01	-.1171E-03	.7376E-02	-.6676E-02	.00

STRUCTURE w/ PILE REACTIONS FROM CPGA.

LOAD CASE 1 ; 1 FT. STRIP

ACTUAL LOAD WIDTH = 2.5 + 1.5 = 4.0'



PILE NUMBER	32	31	30	29
PILE REACTION PER FT WIDTH	13.6 k	13.6 k	13.6 k	13.6 k

M = k·FT SEE P. 17

V_1 = STEM WT PLUS DEAD LOAD = 15.7 k

w(NET) = - STILL WATER : 20.4 (.0625) = 1.275 KSF

CONCRETE : 3 (.15) = .45

UPLIFT : = -.90

0.825 KSF

V_2 = PIER LOAD = 43.3 k

HALF THE STRUCTURE: INVESTIGATE SLAB LOADS

1. $\Sigma F_y = 15.7 + .825(24.5) + 43.3 - 4(13.6) = 24.8$ k

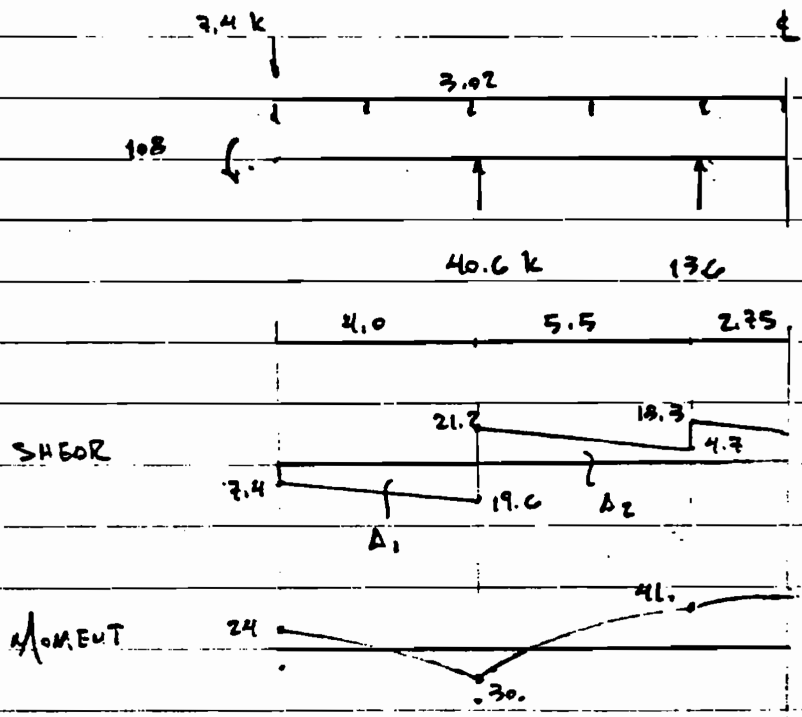
SUBTRACT $24.8/3 = 8.3$ k FROM V_1, V_2

$V_1 = 15.7 - 8.3 = 7.4$ k

$V_2 = 43.3 - 8.3 = 35.0$ k

2. INCREASE REACTION AT PILE 32: 108 k·FT $/ 4 = 27.$ k

INCREASE DISTRIBUTED LOAD BY ADDITIONAL
$27 / [24.5/2] = 2.2$ k/FT
$.825 + 2.2 = 3.02$ k/FT



$\Delta_1 = 7.4(4) + .5(19.6 - 7.4)(4)$
$= 29.6 + 24.4 = 54.0$

$\Delta_2 = 4.7(5.5) + .5(21.2 - 4.7)(5.5)$
$= 25.8 + 45.4 = 71.2$

FLEXURE : SLAB

1. MEMBER CAPACITY BASED ON $\rho = .25\rho_b$

$\phi M_n = 3220$ k.IN SEE P.7

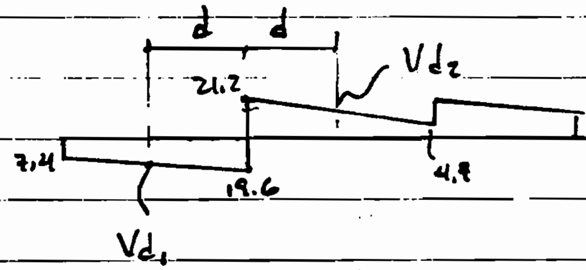
CFRAME @ MEM 3

2. $M_u = 1.3(1.7)(596) = 1317$ k.IN

3. $\phi M_n > M_u$ OK

SHEAR : SLAB

A. FLEXURAL SHEAR



$Vd_1 \approx Vd_2 = 21.2 - \left(\frac{31.5}{66}\right)(21.2 - 4.7) = 13.3$ k

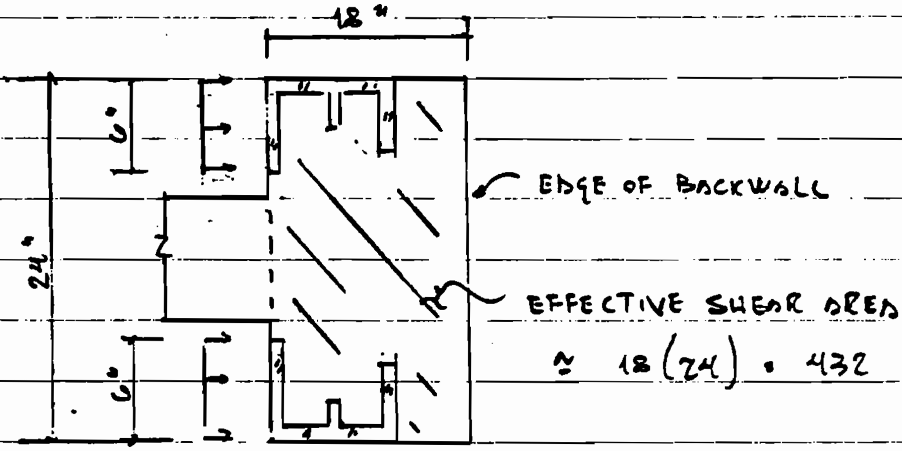
1. $V_u = 1.3(1.7)(13.3) = 29.4$ k

2. $\phi V_c = 35.1$ k P.12

3. $\phi V_c > V_u$ NO SHEAR REINFORCEMENT

B. PIER BETWEEN GATES AND SLAB.

SHEAR AT BASE OF PIER



$\approx 18(24) = 432 \text{ in}^2$

LOAD FROM GATE GUIDES:

$2(.0625)(22.9)(8/2) = 11.45 \text{ k}$

HALF GATE WIDTH

$V_u = 1.3(1.7)(11.45) = 25.3 \text{ k}$

SHEAR FRICTION:

$\phi V_n = \phi A_{vf} f_y \mu$

A_{vf} = USE TEMP. STEEL = $.0028(432) = 1.20 \text{ in}^2$

μ = USE INTENTIONALLY ROUGHENED SURFACE

$= 1.0\lambda = 1.(1) = 1$

$\phi V_n = .85(1.2)(60)(1) = 61.2 \text{ k}$

$\phi V_n > V_u$ OK

C. BACK WALL : SEE GATED MONOLITH CALCS.